



MATERIALS FOR CIVIL AND CONSTRUCTION ENGINEERS

SECOND EDITION

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PREFACE

A basic function of civil and construction engineering is to provide and maintain the infrastructure needs of society. The infrastructure includes buildings, water treatment and distribution systems, waste water removal and processing, dams, and highway and airport bridges and pavements. Although some civil and construction engineers are involved in the planning process, most are concerned with the design, construction, and maintenance of facilities. The common denominator among these responsibilities is the need to understand the behavior and performance of materials. Although not all civil and construction engineers need to be material specialists, a basic understanding of the material selection process, and the behavior of materials, is a fundamental requirement for all civil and construction engineers performing design, construction, and maintenance.

Material requirements in civil engineering and construction facilities are different from material requirements in other engineering disciplines. Frequently, civil engineering structures require tons of materials with relatively low replications of specific designs. Generally, the materials used in civil engineering have relatively low unit costs. In many cases, civil engineering structures are formed or fabricated in the field under adverse conditions. Finally, many civil engineering structures are directly exposed to detrimental effects of the environment.

The subject of engineering materials has advanced greatly in the last few decades. As a result, many of the conventional materials have either been replaced by more efficient materials or modified to improve their performance. Civil and construction engineers have to be aware of these advances and be able to select the most cost-effective material or use the appropriate modifier for the specific application at hand.

This text is organized into three parts: (1) introduction to materials engineering, (2) characteristics of materials used in civil and construction engineering, and (3) laboratory methods for the evaluation of materials.

The introduction to materials engineering includes information on the basic mechanistic properties of materials, environmental influences, and basic material classes. In addition, one of the responsibilities of civil and construction engineers is the inspection and quality control of materials in the construction process. This requires an understanding of material variability and testing procedures. The atomic structure of materials is covered in order to provide basic understanding of material behavior and to relate the molecular structure to the engineering response.

The second section, which represents a large portion of the book, presents the characteristics of the primary material types used in civil and construction engineering: steel, aluminum, concrete, masonry, asphalt, and wood. Since the discussion of concrete and asphalt materials requires a basic knowledge of aggregates, there is a chapter on aggregates. Moreover, since composites are gaining wide acceptance among engineers and are replacing many of the conventional materials, there is a chapter introducing composites.

The discussion of each type of material includes information on the following:

- Basic structure of the materials
- Material production process
- Mechanistic behavior of the material and other properties
- Environmental influences
- Construction considerations
- Special topics related to the material discussed in each chapter

Finally, each chapter includes an overview of various test procedures to introduce the test methods used with each material. However, the detailed description of the test procedures is left to the appropriate standards organizations such as the American Society for Testing and Materials (ASTM) and the American Association of State Highway and Transportation Officials (AASHTO). These ASTM and AASHTO standards are usually available in college libraries, and students are encouraged to use them. Also, there are sample problems in most chapters, as well as selected questions and problems at the end of each chapter. Answering these questions and problems will lead to a better understanding of the subject matter.

There are volumes of information available for each of these materials. It is not possible, or desirable, to cover these materials exhaustively in an introductory single text. Instead, this book limits the information to an introductory level, concentrates on current practices, and extracts information that is relevant to the general education of civil and construction engineers.

The content of the book is intended to be covered in one academic semester, although quarter system courses can definitely use it. The instructor of the course can also change the emphasis of some topics to match the specific curriculum of the department. Furthermore, since the course usually includes a laboratory portion, a number of laboratory test methods are described. The number of laboratory tests in the book is more than what is needed in a typical semester in order to provide more flexibility to the instructor to use the available equipment. Laboratory tests should be coordinated with the topics covered in the lectures so that the students get the most benefit from the laboratory experience.

The first edition of this textbook seemed to serve the needs of many universities and colleges. Therefore, the second edition is more of a refinement and updating of the book, with some notable additions. Several edits were made to the steel chapter to improve the description of heat treatments, phase diagram, and the heat-treating effects of welding. Also, a section on stainless steel was added, and current information on the structural uses of

steel was provided. The cement and concrete chapters have been augmented with sections on hydration-control admixtures, recycled wash water, silica fume, self-consolidating concrete, and flowable fill. When the first edition was published, the Superpave mix design method was just being introduced to the industry. Now Superpave is a well-established method that has been field tested and revised to better meet the needs of the paving community. This development required a complete revision to the asphalt chapter to accommodate the current methods and procedures for both Performance Grading of asphalt binders and the Superpave mix design method. The chapter on wood was revised to provide information on recent manufactured wood products that became available in the last several years. Also, since fiber reinforced polymer composites have been more commonly used in retrofitting old and partially damaged structures, several examples were added in the chapter on composites. In the laboratory manual, an experiment on dry-rodded unit weight of aggregate that is used in portland cement concrete (PCC) proportioning was added and the experiment on creep of asphalt concrete was deleted for lack of use.

In addition to the technical content revisions, there are over 100 new figures to display concepts and equipment. Multiple sample problems and homework problems have been added to each chapter to allow professors to vary assignments between semesters.

The authors would like to acknowledge the contributions of Drs. Barzin Mobasher and Chris Lawrence of Arizona State University, Mr. Jim Willson and Mr. Paul Mueller of the Portland Cement Association, Dr. Mansour Solimanian of Pennsylvania State University, Mr. Lary Lenke of the University of New Mexico, and Dr. Nabil Grace of Lawrence Tech University for their advice and for providing some photos and homework problems. Appreciation also goes to Mr. Serif El-Badawy of Arizona State University for his contribution in the preparation of the solutions manual.

1

MATERIALS ENGINEERING CONCEPTS



Materials engineers are responsible for the selection, specification, and quality control of materials to be used in a job. These materials must meet certain classes of criteria or materials properties (Ashby and Jones 1980). These classes of criteria include

- economic factors
- mechanical properties
- nonmechanical properties
- production/construction considerations
- aesthetic properties

When engineers select the material for a specific application, they must consider the various criteria and make compromises. Both the client and the purpose of the facility or structure dictate, to a certain extent, the emphasis that will be placed on the different criteria.

Civil and construction engineers must be familiar with materials used in the construction of a wide range of structures. Materials most frequently used include steel, aggregate, concrete, masonry, asphalt, and wood. Materials used to a lesser extent include aluminum, glass, plastics, and fiber-reinforced composites. Geotechnical engineers make a reasonable case for including soil as the most widely used engineering material, since it provides the basic support for all civil engineering structures. However, the properties of soils will not be discussed in this text, because this is generally the topic of a separate course.

Recent advances in the technology of civil engineering materials have resulted in the development of better quality, more economical, and safer materials. These materials are commonly referred to as high-performance materials. Because more is known about the molecular structure of materials and because of the continuous research efforts by scientists and engineers, new materials such as polymers, adhesives, composites, geotextiles, coatings, cold-formed metals, and various synthetic products are competing with traditional civil engineering materials. In addition, improvements have

been made to existing materials by changing their molecular structures or including additives to improve quality, economy, and performance. For example, superplasticizers have made a breakthrough in the concrete industry, allowing the production of much stronger concrete. Joints made of elastomeric materials have improved the safety of high-rise structures in earthquake-active areas. Lightweight synthetic aggregates have decreased the weight of concrete structures, allowing small cross-sectional areas of components. Polymers have been mixed with asphalt, allowing pavements to last longer under the effect of vehicle loads and environmental conditions.

The field of fiber composite materials has developed rapidly in the last 30 years. Many recent civil engineering projects have used fiber-reinforced composites. These advanced composites compete with traditional materials due to their higher strength-to-weight ratio and their ability to overcome such shortcomings as corrosion. For example, fiber-reinforced concrete has much greater toughness than conventional portland cement concrete. Composites can replace reinforcing steel in concrete structures. In fact, composites have allowed the construction of structures that could not have been built in the past.

The nature and behavior of civil engineering materials are as complicated as those of materials used in any other field of engineering. Due to the high quantity of materials used in civil engineering projects, the civil engineer frequently works with locally available materials that are not as highly refined as the materials used in other engineering fields. As a result, civil engineering materials frequently have highly variable properties and characteristics.

This chapter reviews the manner in which the properties of materials affect their selection and performance in civil engineering applications. In addition, the chapter reviews some basic definitions and concepts of engineering mechanics required for understanding material behavior. The variable nature of material properties is also discussed so that the engineer will understand the concepts of precision and accuracy, sampling, quality assurance, and quality control. Finally, instruments used for measuring material response are described.

1.1 Economic Factors

The economics of the material selection process are affected by much more than just the cost of the material. Factors that should be considered in the selection of the material include

- availability and cost of raw materials
- manufacturing costs
- transportation
- placing
- maintenance

The materials used for civil engineering structures have changed over time. Early structures were constructed of stone and wood. These materials

were in ready supply and could be cut and shaped with available tools. Later, cast iron was used, because mills were capable of crudely refining iron ore. As the industrial revolution took hold, quality steel could be produced in the quantities required for large structures. In addition, portland cement, developed in the mid-1800s, provided civil engineers with a durable inexpensive material with broad applications.

Due to the efficient transportation system in the United States, availability is not as much of an issue as it once was in the selection of a material. However, transportation can significantly add to the cost of the materials at the job site. For example, in many locations in the United States, quality aggregates for concrete and asphalt are in short supply. The closest aggregate source to Houston, Texas, is 150 km (90 miles) from the city. This haul distance approximately doubles the cost of the aggregates in the city and hence puts concrete at a disadvantage compared with steel.

The type of material selected for a job can greatly affect the ease of construction and the construction costs and time. For example, the structural members of a steel-frame building can be fabricated in a shop, transported to the job site, lifted into place with a crane, and bolted or welded together. In contrast, for a reinforced concrete building, the forms must be built; reinforcing steel placed; concrete mixed, placed, and allowed to cure; and the forms removed. Constructing the concrete frame building can be more complicated and time consuming than constructing steel structures. To overcome this shortcoming, precast concrete units commonly have been used, especially for bridge construction.

All materials deteriorate over time and with use. This deterioration affects both the maintenance cost and the useful life of the structure. The rate of deterioration varies among materials. Thus, in analyzing the economic selection of a material, the life cycle cost should be evaluated in addition to the initial costs of the structure.

1.2 Mechanical Properties

The mechanical behavior of materials is the response of the material to external loads. All materials deform in response to loads; however, the specific response of a material depends on its properties, the magnitude and type of load, and the geometry of the element. Whether the material “fails” under the load conditions depends on the failure criterion. Catastrophic failure of a structural member, resulting in the collapse of the structure, is an obvious material failure. However, in some cases the failure is more subtle, but with equally severe consequences. For example, pavement may fail due to excessive roughness at the surface, even though the stress levels are well within the capabilities of the material. A building may have to be closed due to excessive vibrations by wind or other live loads, although it could be structurally sound. These are examples of *functional* failures.

1.2.1 Loading Conditions

One of the considerations in the design of a project is the type of loading the structure will be subjected to during its design life. The two basic types of loads are static and dynamic. Each type affects the material differently, and frequently the interactions between the load types are important. Civil engineers encounter both when designing a structure.

Static loading implies a sustained loading of the structure over a period of time. Generally, static loads are slowly applied such that no shock or vibration is generated in the structure. Once applied, the static load may remain in place or be removed slowly. Loads that remain in place for an extended period of time are called *sustained* (dead) loads. In civil engineering, much of the load the materials must carry is due to the weight of the structure and equipment in the structure.

Loads that generate a shock or vibration in the structure are *dynamic* loads. Dynamic loads can be classified as *periodic*, *random*, or *transient*, as shown in Figure 1.1 (Richart et al. 1970). A periodic load, such as a harmonic or sinusoidal load, repeats itself with time. For example, rotating equipment in a building can produce a vibratory load. In a random load, the

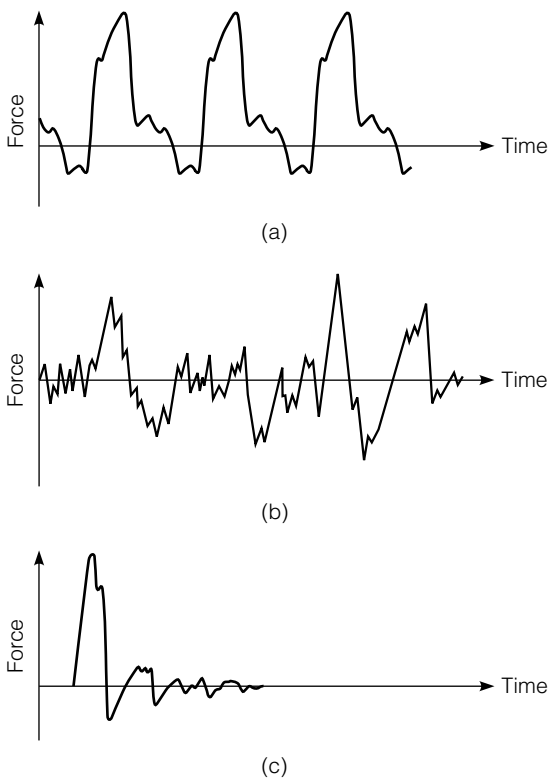


FIGURE 1.1 Types of dynamic loads: (a) periodic, (b) random, and (c) transient.

load pattern never repeats, such as that produced by earthquakes. Transient load, on the other hand, is an impulse load that is applied over a short time interval, after which the vibrations decay until the system returns to a rest condition. For example, bridges must be designed to withstand the transient loads of trucks.

1.2.2 ■ Stress–Strain Relations

Materials deform in response to loads or forces. In 1678, Robert Hooke published the first findings that documented a linear relationship between the amount of force applied to a member and its deformation. The amount of deformation is proportional to the properties of the material and its dimensions. The effect of the dimensions can be normalized. Dividing the force by the cross-sectional area of the specimen normalizes the effect of the loaded area. The force per unit area is defined as the stress σ in the specimen (i.e., $\sigma = \text{Force}/\text{Area}$). Dividing the deformation by the original length is defined as strain ε of the specimen (i.e., $\varepsilon = \text{Change in length}/\text{Original length}$). Much useful information about the material can be determined by plotting the stress–strain diagram.

Figure 1.2 shows typical uniaxial tensile or compressive stress–strain curves for several engineering materials. Figure 1.2(a) shows a linear stress–strain relationship up to the point where the material fails. Glass and chalk are typical of materials exhibiting this tensile behavior. Figure 1.2(b) shows the behavior of steel in tension. Here, a linear relationship is obtained up to a certain point (proportional limit), after which the material deforms without much increase in stress. On the other hand, aluminum alloys in tension exhibit a linear stress–strain relation up to the proportional limit, after which a nonlinear relation follows, as illustrated in Figure 1.2(c). Figure 1.2(d) shows a nonlinear relation throughout the whole range. Concrete and other materials exhibit this relationship, although the first portion of the curve for concrete is very close to being linear. Soft rubber in tension differs from most materials in such a way that it shows an almost linear stress–strain relationship followed by a reverse curve, as shown in Figure 1.2(e).

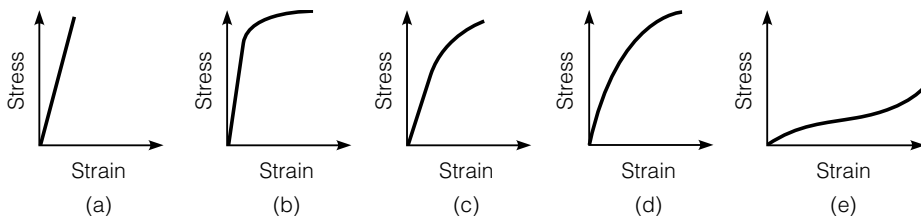


FIGURE 1.2 Typical uniaxial stress–strain diagrams for some engineering materials: (a) glass and chalk, (b) steel, (c) aluminum alloys, (d) concrete, and (e) soft rubber.

1.2.3 ■ Elastic Behavior

If a material exhibits true elastic behavior, it must have an instantaneous response (deformation) to load, and the material must return to its original shape when the load is removed. Many materials, including most metals, exhibit elastic behavior, at least at low stress levels. As will be discussed in Chapter 2, elastic deformation does not change the arrangement of atoms within the material, but rather it stretches the bonds between atoms. When the load is removed the atomic bonds return to their original position.

Young observed that different elastic materials have different proportional constants between stress and strain. For a homogeneous, isotropic, and linear elastic material, the proportional constant between normal stress and normal strain of an axially loaded member is the *modulus of elasticity* or *Young's modulus*, E , and is equal to

$$E = \frac{\sigma}{\varepsilon} \quad (1.1)$$

where σ is the normal stress and ε is the normal strain.

In the axial tension test, as the material is elongated, there is a reduction of the cross section in the lateral direction. In the axial compression test, the opposite is true. The ratio of the lateral strain, ε_l , to the axial strain, ε_a , is *Poisson's ratio*,

$$\nu = \frac{-\varepsilon_l}{\varepsilon_a} \quad (1.2)$$

Since the axial and lateral strains will always have different signs, the negative sign is used in Equation 1.2 to make the ratio positive. Poisson's ratio has a theoretical range of 0.0 to 0.5, where 0.0 is for a compressible material in which the axial and lateral directions are not affected by each other. The 0.5 value is for a material that does not change its volume when the load is applied. Most solids have Poisson's ratios between 0.10 and 0.45.

Although Young's modulus and Poisson's ratio were defined for the uniaxial stress condition, they are important when describing the three-dimensional stress–strain relationships, as well. If a homogeneous, isotropic cubical element with linear elastic response is subjected to normal stresses σ_x , σ_y , and σ_z in the three orthogonal directions (as shown in Figure 1.3),

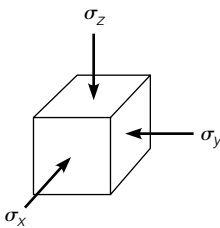


FIGURE 1.3 Normal stresses applied on a cubical element.

the normal strains ε_x , ε_y , and ε_z can be computed by the *generalized Hooke's law*,

$$\begin{aligned}\varepsilon_x &= \frac{\sigma_x - \nu(\sigma_y + \sigma_z)}{E} \\ \varepsilon_y &= \frac{\sigma_y - \nu(\sigma_z + \sigma_x)}{E} \\ \varepsilon_z &= \frac{\sigma_z - \nu(\sigma_x + \sigma_y)}{E}\end{aligned}\quad (1.3)$$

Sample Problem 1.1

A cube made of an alloy with dimensions of 50 mm \times 50 mm \times 50 mm is placed into a pressure chamber and subjected to a pressure of 90 MPa. If the modulus of elasticity of the alloy is 100 GPa and Poisson's ratio is 0.28, what will be the length of each side of the cube, assuming that the material remains within the elastic region?

Solution

$$\begin{aligned}\varepsilon_x &= [\sigma_x - \nu(\sigma_y + \sigma_z)]/E = [-90 - 0.28 \times (-90 - 90)]/100,000 \\ &= -0.000396 \text{ m/m} \\ \varepsilon_y &= \varepsilon_z = -0.000396 \text{ m/m} \\ \Delta x &= \Delta y = \Delta z = -0.000396 \times 50 = -0.0198 \text{ mm} \\ L_{\text{new}} &= 50 - 0.0198 = 49.9802 \text{ mm}\end{aligned}$$

Linearity and elasticity should not be confused. A *linear material's* stress–strain relation follows a straight line. An *elastic material* returns to its original shape when the load is removed and reacts instantaneously to changes in load. For example, Figure 1.4(a) represents a linear elastic behavior, while Figure 1.4(b) represents a nonlinear elastic behavior.

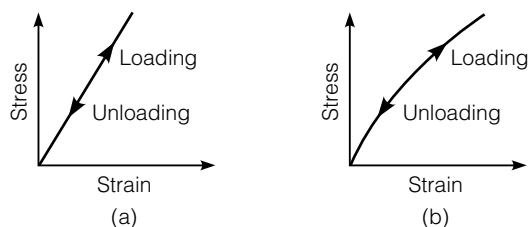


FIGURE 1.4 Elastic behavior: (a) linear and (b) nonlinear.

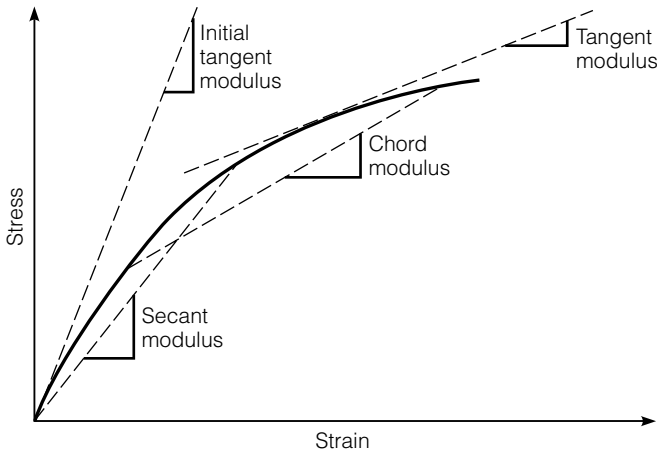


FIGURE 1.5 Methods for approximating modulus.

For materials that do not display any linear behavior, such as concrete and soils, determining a Young's modulus or elastic modulus can be problematical. There are several options for arbitrarily defining the modulus for these materials. Figure 1.5 shows four options: the initial tangent, tangent, secant, and chord moduli. The *initial tangent modulus* is the slope of the tangent of the stress–strain curve at the origin. The *tangent modulus* is the slope of the tangent at a point on the stress–strain curve. The *secant modulus* is the slope of a chord drawn between the origin and an arbitrary point on the stress–strain curve. The *chord modulus* is the slope of a chord drawn between two points on the stress–strain curve. The selection of which modulus to use for a nonlinear material depends on the stress or strain level at which the material typically is used. Also, when determining the tangent, secant, or chord modulus, the stress or strain levels must be defined.

Table 1.1 shows typical modulus and Poisson's ratio values for some materials at room temperature. Note that some materials have a range of modulus values rather than a distinct value. Several factors affect the modulus, such as curing level and proportions of components of concrete or the direction of loading relative to the grain of wood.

1.2.4 ■ Elastoplastic Behavior

For some materials, as the stress applied on the specimen is increased, the strain will proportionally increase up to a point; after this point the strain will increase with little additional stress. In this case, the material exhibits linear elastic behavior followed by plastic response. The stress level at which the behavior changes from elastic to plastic is the *elastic limit*. When the load is removed from the specimen, some of the deformation will be recovered and some of the deformation will remain as seen in Figure 1.6(a). As discussed in Chapter 2, plastic behavior indicates permanent deformation of the specimen so that it does not return to its original shape when the load is removed. This indicates that when the load is applied, the atomic bonds

TABLE 1.1 Typical Modulus and Poisson's Ratio Values (Room Temperature)

Material	Modulus GPa (psi $\times 10^6$)	Poisson's Ratio
Aluminum	69–75 (10–11)	0.33
Brick	10–17 (1.5–2.5)	0.23–0.40
Cast iron	75–169 (11–23)	0.17
Concrete	14–40 (2–6)	0.11–0.21
Copper	110 (16)	0.35
Epoxy	3–140 (0.4–20)	
Glass	62–70 (9–10)	0.25
Limestone	58 (8.4)	
Rubber (soft)	0.001–0.014 (0.00015–0.002)	0.49
Steel	207 (30)	0.27
Tungsten	407 (59)	0.28
Wood	6–15 (0.9–2.2)	

stretch, creating an elastic response; then the atoms actually slip relative to each other. When the load is removed, the atomic slip does not recover; only the atomic stretch is recovered (Callister 2003).

Several models are used to represent the behavior of materials that exhibit both elastic and plastic responses. Figure 1.6(b) shows a linear elastic–perfectly plastic response in which the material exhibits a linear elastic response upon loading, followed by a completely plastic response. If such material is unloaded after it has plastically deformed, it will rebound in a linear elastic manner and will follow a straight line parallel to the elastic portion, while some permanent deformation will remain. If the material is loaded again, it will have a linear elastic response followed by plastic response at the same level of stress at which the material was unloaded (Popov 1968).

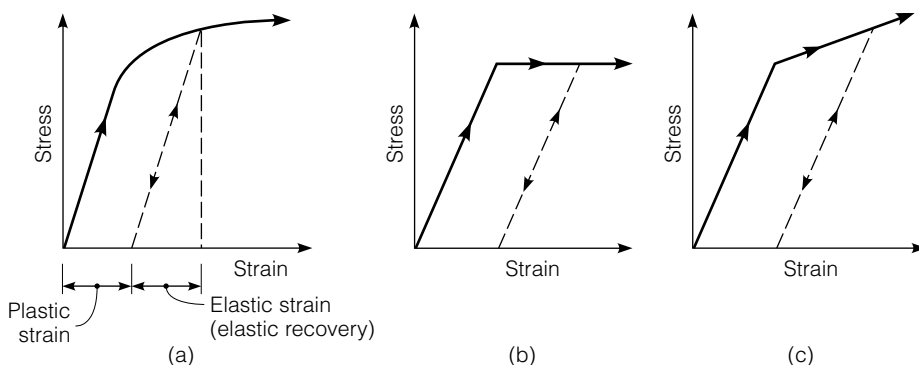
**FIGURE 1.6** Stress–strain behavior of plastic materials: (a) example of loading and unloading, (b) elastic–perfectly plastic, and (c) elastoplastic with strain hardening.

Figure 1.6(c) shows an elastoplastic response in which the first portion is an elastic response followed by a combined elastic and plastic response. If the load is removed after the plastic deformation, the stress–strain relation will follow a straight line parallel to the elastic portion; consequently, some of the strain in the material will be removed, and the remainder of the strain will be permanent. Upon reloading, the material again behaves in a linear elastic manner up to the stress level that was attained in the previous stress cycle. After that point the material will follow the original stress–strain curve. Thus, the stress required to cause plastic deformation actually increases. This process is called *strain hardening* or *work hardening*. Strain hardening is beneficial in some cases, since it allows more stress to be applied without permanent deformation. Mild steel is an example of material that experiences strain hardening during plastic deformation.

Some materials exhibit *strain softening*, in which plastic deformation causes weakening of the material. Portland cement concrete is a good example of such a material. In this case, plastic deformation causes microcracks at the interface between aggregate and cement paste.

Sample Problem 1.2

An elastoplastic material with strain hardening has the stress–strain relation shown in Figure 1.6(c). The modulus of elasticity is 25×10^6 psi, yield strength is 70 ksi, and the slope of the strain-hardening portion of the stress–strain diagram is 3×10^6 psi.

- Calculate the strain that corresponds to a stress of 80 ksi.
- If the 80-ksi stress is removed, calculate the permanent strain.

Solution

$$\begin{aligned} \text{(a) } \epsilon &= (70,000/25 \times 10^6) + [(80,000 - 70,000)/(3 \times 10^6)] = 0.0028 + 0.0033 \\ &= 0.0061 \text{ in./in.} \end{aligned}$$

$$\begin{aligned} \text{(b) } \epsilon_{\text{permanent}} &= 0.0061 - [80,000/(25 \times 10^6)] = 0.0061 - 0.0032 \\ &= 0.0029 \text{ in./in.} \end{aligned}$$

Materials that do not undergo plastic deformation prior to failure, such as concrete, are said to be *brittle*, whereas materials that display appreciable plastic deformation, such as mild steel, are *ductile*. Generally, ductile materials are preferred for construction. When a brittle material fails, the structure can collapse in a catastrophic manner. On the other hand, overloading a ductile material will result in distortions of the structure, but the structure will not necessarily collapse. Thus, the ductile material provides the designer with a margin of safety.

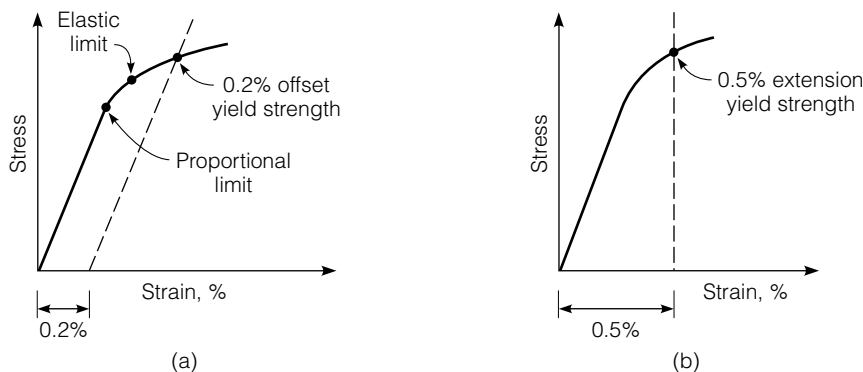


FIGURE 1.7 Methods for estimating yield stress: (a) offset method and (b) extension method.

Figure 1.7(a) demonstrates three concepts of the stress–strain behavior of elastoplastic materials. The lowest point shown on the diagram is the *proportional limit*, defined as the transition point between linear and nonlinear behavior. The second point is the *elastic limit*, which is the transition between elastic and plastic behavior. However, most materials do not display an abrupt change in behavior from elastic to plastic. Rather, there is a gradual, almost imperceptible transition between the behaviors, making it difficult to locate an exact transition point (Polowski and Ripling 1966). For this reason, arbitrary methods such as the *offset* and the *extension* methods, are used to identify the elastic limit, thereby defining the *yield stress (yield strength)*. In the offset method, a specified offset is measured on the abscissa, and a line with a slope equal to the initial tangent modulus is drawn through this point. The point where this line intersects the stress–strain curve is the *offset yield stress* of the material, as seen in Figure 1.7(a). Different offsets are used for different materials (Table 1.2). The *extension yield*

TABLE 1.2 Offset Values Typically Used to Determine Yield Stress

Material	Stress Condition	Offset (%)	Corresponding Strain
Steel	Tension	0.20	0.0020
Wood	Compression parallel to grain	0.05	0.0005
Gray cast iron	Tension	0.05	0.0005
Concrete	Compression	0.02	0.0002
Aluminum alloys	Tension	0.20	0.0020
Brass and bronze	Tension	0.35	0.0035

stress is located where a vertical projection, at a specified strain level, intersects the stress–strain curve. Figure 1.7(b) shows the yield stress corresponding to 0.5% extension.

Sample Problem 1.3

A rod made of aluminum alloy, with a gauge length of 100 mm, diameter of 10 mm, and yield strength of 150 MPa, was subjected to a tensile load of 5.85 kN. If the gauge length was changed to 100.1 mm and the diameter was changed to 9.9967 mm, calculate the modulus of elasticity and Poisson's ratio.

Solution

$$\sigma = P/A = (5850 \text{ N})/[\pi(5 \times 10^{-3} \text{ m})^2] = 74.5 \times 10^6 \text{ Pa} = 74.5 \text{ MPa}$$

Since the applied stress is well below the yield strength, the material is within the elastic region.

$$\varepsilon_a = \Delta L/L = (100.1 - 100)/100 = 0.001$$

$$E = \sigma/\varepsilon_a = 74.5/0.001 = 74,500 \text{ MPa} = 74.5 \text{ GPa}$$

$$\varepsilon_l = \text{change in diameter/diameter} = (9.9967 - 10)/10 = -0.00033$$

$$\nu = -\varepsilon_l/\varepsilon_a = 0.00033/0.001 = 0.33$$

1.2.5 Work and Energy

When a material is tested, the testing machine is actually generating a force in order to move or deform the specimen. Since work is force times distance, the area under a force–displacement curve is the work done on the specimen. When the force is divided by the cross-sectional area of the specimen to compute the stress, and the deformation is divided by the length of the specimen to compute the strain, the force–displacement diagram becomes a stress–strain diagram. However, the area under the stress–strain diagram no longer has the units of work. By manipulating the units of the stress–strain diagram, we can see that the area under the stress–strain diagram equals the work per unit volume of material required to deform or fracture the material. This is a useful concept, for it tells us the energy that is required to deform or fracture the material. Such information is used for selecting materials to use where energy must be absorbed by the member. The area under the elastic portion of the curve is the *modulus of resilience* [Figure 1.8(a)]. The amount of energy required to fracture a specimen is a measure of the *toughness* of the material, as in Figure 1.8(b). As shown in Figure 1.8(c), a high-strength material is not necessarily a tough material.

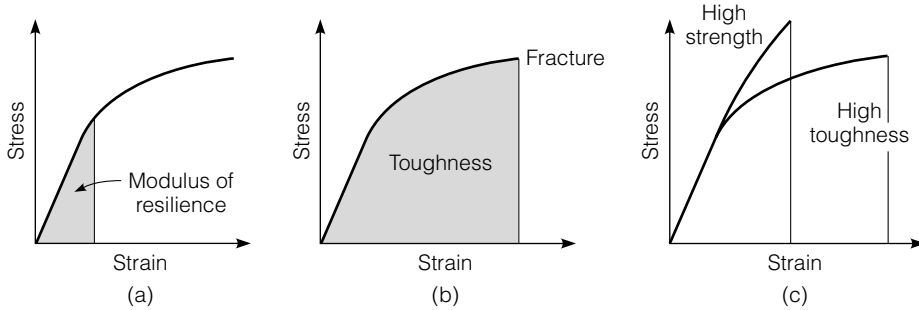


FIGURE 1.8 Areas under stress–strain curves: (a) modulus of resilience, (b) toughness, and (c) high-strength and high-toughness materials.

For instance, as will be discussed in Chapter 3, increasing the carbon content of steel increases the yield strength, but reduces ductility. Therefore, the strength is increased, but the toughness may be reduced.

1.2.6 ■ Time-Dependent Response

The previous discussion assumed that the strain was an immediate response to stress. This is an assumption for elastic materials. However, no material has this property under all conditions. In some cases, materials have a delayed response. The amount of deformation depends on the duration of the load, the temperature, and the material characteristics. There are several mechanisms associated with time-dependent deformation, such as *creep* and *viscous flow*. There is no clear distinction between these terms. Creep is generally associated with long-term deformations and can occur in metals, ionic and covalent crystals, and amorphous materials. On the other hand, viscous flow is associated only with amorphous materials and can occur under short-term load duration. For example, concrete, a material with predominantly covalent crystals, can creep over a period of decades. Asphalt concrete pavements, an amorphous-binder material, can have ruts caused by the accumulated effect of viscous flows resulting from traffic loads with a load duration of only a fraction of a second.

Creep of metals is not relevant in typical civil engineering applications. In steel, creep can occur at temperatures greater than 30% of the melting point on the absolute scale. This may be a concern in the design of boilers and nuclear reactor containment vessels. Creep of metals occurs in three phases. The first phase is the result of dislocation movements in the molecular structure of the metal. The second phase is associated with slip at the grain boundaries, similar to plastic deformation, but accelerated due to the high temperature. The third phase is associated with an increase in the strain due to a reduction of the cross section of the specimen. Creep is also considered in the design of wood and advanced composite structural members.

Wood elements loaded for a few days can carry higher stresses than elements designed to carry “permanent” loads. On the other hand, creep of concrete is associated with microcracking at the interface of the cement paste and the aggregate particles (Mehta and Monteiro 1993).

The viscous flow models are similar in nature to Hooke’s law. In linearly viscous materials, the rate of deformation is proportional to the stress level. These materials are not compressible and do not recover when the load is removed. Materials with these characteristics are *Newtonian fluids*.

Figure 1.9(a) shows a typical creep test in which a constant compressive stress is applied to an asphalt concrete specimen. In this case, an elastic strain will develop, followed by time-dependent strain or creep. If the specimen is unloaded, a part of the strain will recover instantaneously, while the remaining strain will recover, either completely or partially, over a period of time. Another phenomenon typical of time-dependent materials is relaxation, or dissipation of stresses with time. For example, if an asphalt concrete specimen is placed in a loading machine and subjected to a constant strain, the stress within the specimen will initially be high, then gradually dissipate due to relaxation as shown in Figure 1.9(b). Relaxation is an important concern in the selection of steel for a prestressed concrete design.

Materials exhibiting both viscous and elastic responses are known as *viscoelastic*. In viscoelasticity, there are two approaches used to describe how stresses, strains, and time are interrelated. One approach is to postulate mathematical relations between these parameters based on material functions obtained from laboratory tests. The other approach is based on combining a number of discrete *rheological elements* to form *rheological models*, which describe the material response.

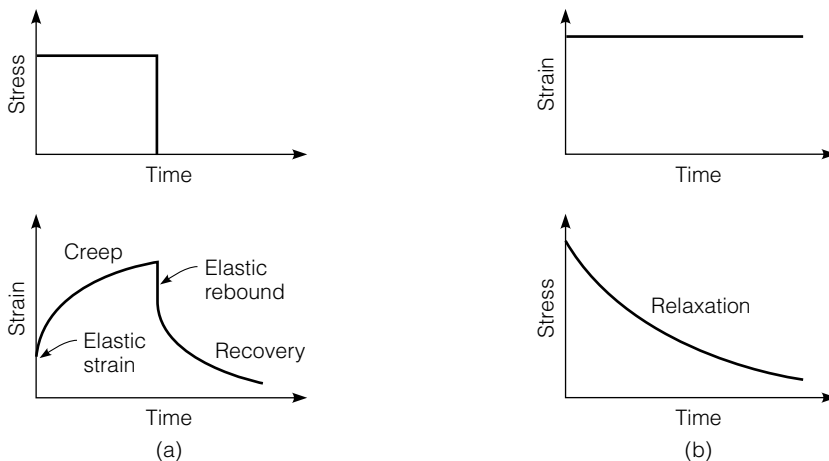


FIGURE 1.9 Behavior of time-dependent materials: (a) creep and (b) relaxation.

1.2.7 ■ Rheological Models

Rheological models are used to model mechanically the time-dependent behavior of materials. There are many different modes of material deformation, particularly in polymer materials. These materials cannot be described as simply elastic, viscous, etc. However, these materials can be modeled by a combination of simple physical elements. The simple physical elements have characteristics that can be easily visualized. Rheology uses three basic elements, combined in either series or parallel to form models that define complex material behaviors. The three basic rheological elements, Hookean, Newtonian, and St. Venant, are shown in Figure 1.10 (Polowski and Rippling 1966).

The *Hookean* element, as in Figure 1.10(a), has the characteristics of a linear *spring*. The deformation δ is proportional to force F by a constant M :

$$F = M\delta \quad (1.4)$$

This represents a perfectly linear elastic material. The response to a force is instantaneous and the deformation is completely recovered when the force is removed. Thus, the Hookean element represents a perfectly linear elastic material.

A *Newtonian* element models a perfectly viscous material and is modeled as a *dashpot* or shock absorber as seen in Figure 1.10(b). The deformation for a given level of force is proportional to the amount of time the force is applied. Hence, the rate of deformation, for a constant force, is a constant β :

$$F = \beta\dot{\delta} \quad (1.5)$$

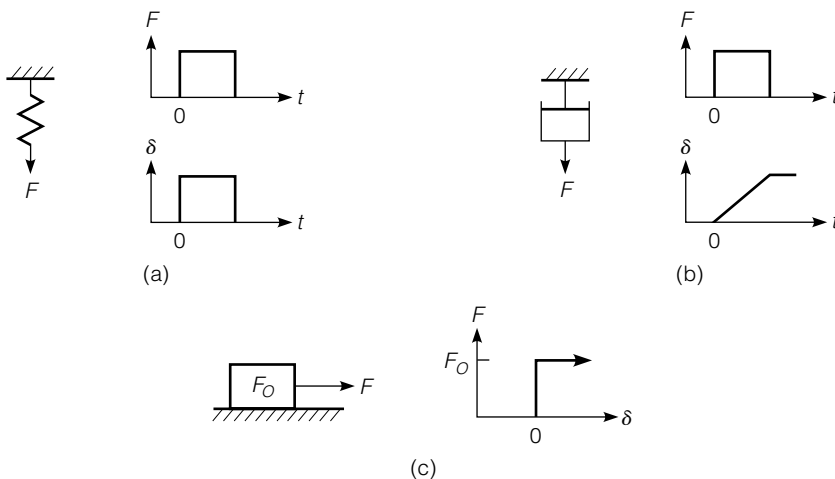


FIGURE 1.10 Basic elements used in rheology: (a) Hookean, (b) Newtonian, and (c) St. Venant.

The dot above the δ defines this as the rate of deformation with respect to time. If $\dot{\delta} = 0$ at time $t = 0$ when a constant force F is applied, the deformation at time t is

$$\delta = \frac{Ft}{\beta} \tag{1.6}$$

When the force is removed, the specimen retains the deformed shape. There is no recovery of any of the deformation.

The *St. Venant* element, as seen in Figure 1.10(c), has the characteristics of a sliding block that resists movement by friction. When the force F applied to the element is less than the critical force F_O , there is no movement. If the force is increased to overcome the static friction, the element will slide and continue to slide as long as the force is applied. This element is unrealistic, since any sustained force sufficient to cause movement would cause the block to accelerate. Hence, the *St. Venant* element is always used in combination with the other basic elements.

The basic elements are usually combined in parallel or series to model material response. Figure 1.11 shows the three primary two-component models: the Maxwell, Kelvin, and Prandtl models. The Maxwell and Kelvin models have a spring and dashpot in series and parallel, respectively. The Prandtl model uses a spring and *St. Venant* elements in series.

In the *Maxwell* model [Figure 1.11(a)], the total deformation is the sum of the deformations of the individual elements. The force in each of the elements must be equal to the total force ($F = F_1 = F_2$). Thus, the equation for the total deformation at any time after a constant load is applied is simply

$$\delta = \delta_1 + \delta_2 = \frac{F}{M} + \frac{Ft}{\beta} \tag{1.7}$$

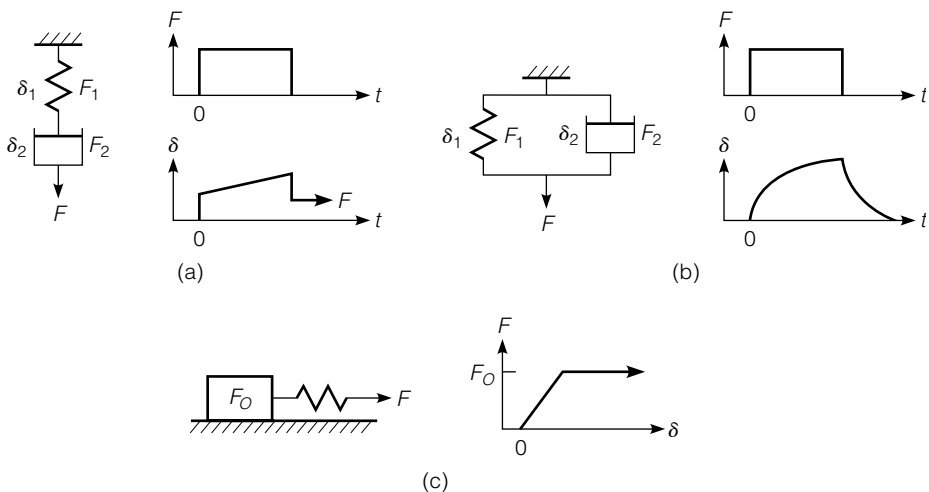


FIGURE 1.11 Two-element rheological models: (a) Maxwell, (b) Kelvin, and (c) Prandtl.

In the *Kelvin* model [Figure 1.11(b)] the deformation of each of the elements must be equal at all times, due to the way the model is formulated. Thus, the total deformation is equal to the deformation of each element ($\delta = \delta_1 = \delta_2$). Since the elements are in parallel, they will share the force such that the total force is equal to the sum of the force in each element. If $\delta = 0$ at time $t = 0$ when a constant force F is applied, Equation 1.4 then requires zero force in the spring. Hence, when the load is initially applied, before any deformation takes place, all of the force must be in the dashpot. Under constant force, the deformation of the dashpot must increase, since there is force on the element. However, this also requires deformation of the spring, indicating that some of the force is carried by the spring. In fact, with time, the amount of force in the dashpot decreases and the force in the spring increases. The proportion is fixed by the fact that the sum on the forces in the two elements must be equal to the total force. After a sufficient amount of time, all of the force will be transferred to the spring and the model will stop deforming. Thus, the maximum deformation of the Kelvin model is $\delta = F/M$. Mathematically, the equation for the deformation in a Kelvin model is derived as

$$F = F_1 + F_2 = M\dot{\delta} + \beta\dot{\delta} \quad (1.8)$$

Integrating Equation 1.8, using the limits that $\delta = 0$ at $t = 0$, and solving for the deformation δ at time t results in

$$\delta = \left(\frac{F}{M}\right)(1 - e^{-Mt/\beta}) \quad (1.9)$$

The *Prandtl* model [Figure 1.11(c)] consists of St. Venant and Hookean bodies in series. The Prandtl model represents a material with an elastic–perfectly plastic response. If a small load is applied, the material responds elastically until it reaches the yield point, after which the material exhibits plastic deformation.

Neither the Maxwell nor Kelvin model adequately describes the behavior of some common engineering materials, such as asphalt concrete. However, the Maxwell and the Kelvin models can be put together in series, producing the *Burgers* model, which can be used to describe simplistically the behavior of asphalt concrete. As shown in Figure 1.12, the Burgers model is generally drawn as a spring in series with a Kelvin model in series

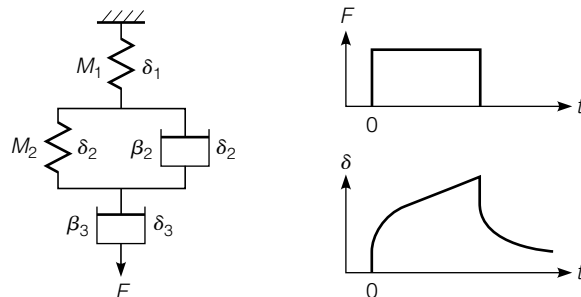


FIGURE 1.12 Burgers model of viscoelastic materials.

with a dashpot. The total deformation, when $\delta = 0$ at time $t = 0$, is then the sum of the deformations of these three elements:

$$\delta = \delta_1 + \delta_2 + \delta_3 = \frac{F}{M_1} + \left(\frac{F}{M_2}\right)(1 - e^{-M_2 t/\beta_2}) + \frac{Ft}{\beta_3} \quad (1.10)$$

The deformation–time diagram for the loading part of the Burgers model demonstrates three distinct phases of behavior. The first phase is the instantaneous deformation of the spring when the load is applied. The second phase is the combined deformation of the Kelvin model and the dashpot. In the third phase, after the Kelvin model reaches maximum deformation, there is a continued deformation of the dashpot at a constant rate of deformation. The unloading part of the Burgers model follows similar behavior.

Some materials require more complicated rheological models to represent their response. In such cases, a number of Maxwell models can be combined in parallel to form the generalized Maxwell model, or a number of Kelvin models in series can be used to form the generalized Kelvin model.

The use of rheological models requires quantifying material parameters associated with each model. Laboratory tests, such as creep tests, can be used to obtain deformation–time curves from which material parameters can be determined.

Fig. SP1.4 the time-dependent response of materials, they can be used only to represent uniaxial responses. The three-dimensional behavior of materials and the Poisson’s effect cannot be represented by these models.

Sample Problem 1.4

Derive the response relation for the model shown in Fig. SP1.4 assuming that the force F is constant and instantaneously applied.

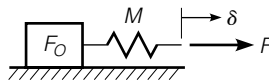


Fig. SP1.4

Solution

$$\text{For } F \leq F_0: \delta = F/M$$

$$\text{For } F > F_0: \text{movement}$$

1.2.8 Temperature and Time Effects

The mechanical behavior of all materials is affected by temperature. Some materials, however, are more susceptible to temperature than others. For example, viscoelastic materials, such as plastics and asphalt, are greatly affected by temperature, even if the temperature is changed by only a few

degrees. Other materials, such as metals or concrete, are less affected by temperatures, especially when they are near ambient temperature.

Ferrous metals, including steel, demonstrate a change from ductile to brittle behavior as the temperature drops below the *transition temperature*. This change from ductile to brittle behavior greatly reduces the toughness of the material. While this could be determined by evaluating the stress–strain diagram at different temperatures, it is more common to evaluate the toughness of a material with an impact test that measures the energy required to fracture a specimen. Figure 1.13 shows how the energy required to fracture a mild steel changes with temperature (Flinn and Trojan 1986). The test results seen in Figure 1.13 were achieved by applying impact forces on bar specimens with a “defect” (a simple V notch) machined into the specimens (ASTM E23). During World War II, many Liberty ships sank because the steel used in the ships met specifications at ambient temperature, but became brittle in the cold waters of the North Atlantic.

In addition to temperature, some materials, such as viscoelastic materials, are affected by the load duration. The longer the load is applied, the larger is the amount of deformation or creep. In fact, increasing the load duration and increasing the temperature cause similar material responses. Therefore, temperature and time can be interchanged. This concept is very useful in running some tests. For example, a creep test on an asphalt concrete specimen can be performed with short load durations by increasing the temperature of the material. A *time–temperature shift factor* is then used to adjust the results for lower temperatures.

Viscoelastic materials are not only affected by the duration of the load, but also by the rate of load application. If the load is applied at a fast rate, the material is stiffer than if the load is applied at a slow rate. For example, if a heavy truck moves at a high speed on an asphalt pavement, no permanent deformation may be observed. However, if the same truck is parked on an asphalt pavement on a hot day, some permanent deformations on the pavement surface may be observed.

1.2.9 ■ Failure and Safety

Failure occurs when a member or structure ceases to perform the function for which it was designed. Failure of a structure can take several modes,

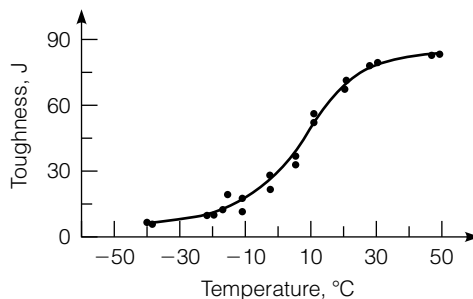


FIGURE 1.13 Fracture toughness of steel under impact testing.

including fracture fatigue, general yielding, buckling, and excessive deformation. *Fracture* is a common failure mode. A brittle material typically fractures suddenly when the static stress reaches the strength of the material, where the strength is defined as the maximum stress the material can carry. On the other hand, a ductile material may fracture due to excessive plastic deformation.

Many structures, such as bridges, are subjected to repeated loadings, creating stresses that are less than the strength of the material. Repeated stresses can cause a material to fail or *fatigue*, at a stress well below the strength of the material. The number of applications a material can withstand depends on the stress level relative to the strength of the material. As shown in Figure 1.14, as the stress level decreases, the number of applications before failure increases. Ferrous metals have an apparent *endurance limit*, or stress level, below which fatigue does not occur. The endurance limit for steels is generally in the range of one-quarter to one-half the ultimate strength (Flinn and Trojan 1986). Another example of a structure that may fail due to fatigue is pavement. Although the stresses applied by traffic are typically much less than the strength of the material, repeated loadings may eventually lead to a loss of the structural integrity of the pavement surface layer, causing fatigue cracks as shown in Figure 1.15.

Another mode of failure is *general yielding*. This failure happens in ductile materials, and it spreads throughout the whole structure, which results in a total collapse.

Long and slender members subjected to axial compression may fail due to *buckling*. Although the member is intended to carry axial compressive loads, a small lateral force might be applied, which causes deflection and eventually might cause failure.

Sometimes *excessive deformation* (elastic or plastic) could be defined as failure, depending on the function of the member. For example, excessive

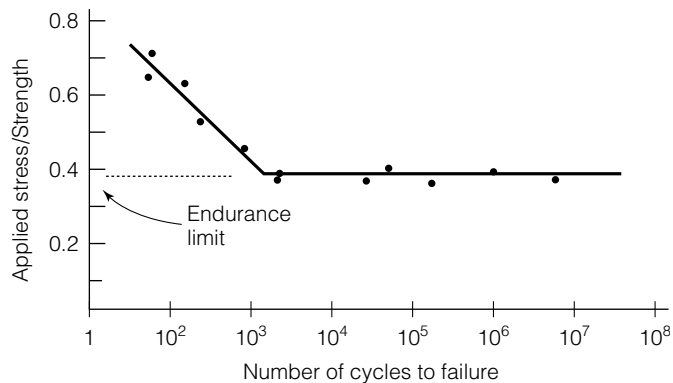


FIGURE 1.14 An example of endurance limit under repeated loading.



FIGURE 1.15 Fatigue failure of asphalt pavement due to repeated traffic loading.

deflections of floors make people uncomfortable and, in an extreme case, may render the building unusable even though it is structurally sound.

To minimize the chance of failure, structures are designed to carry a load greater than the maximum anticipated load. The *factor of safety (FS)* is defined as the ratio of the stress at failure to the allowable stress for design (maximum anticipated stress):

$$FS = \frac{\sigma_{\text{failure}}}{\sigma_{\text{allowable}}} \quad (1.11)$$

where σ_{failure} is the failure stress of the material and $\sigma_{\text{allowable}}$ is the allowable stress for design. Typically, the larger the factor of safety, the larger is the required cross section of the structure and, consequently, the higher is the cost. The proper value of the factor of safety varies from one structure to another and depends on several factors, including the

- cost of unpredictable failure in lives, dollars, and time,
- variability in material properties,
- degree of accuracy in considering all possible loads applied to the structure, such as earthquakes,

- possible misuse of the structure, such as improperly hanging an object from a truss roof,
- degree of accuracy of considering the proper response of materials during design, such as assuming elastic response although the material might not be perfectly elastic.

1.3 Nonmechanical Properties

Nonmechanical properties refer to characteristics of the material, other than load response, that affect selection, use, and performance. There are several types of properties that are of interest to engineers, but those which are of the greatest concern to civil engineers are density, thermal properties, and surface characteristics.

1.3.1 Density and Unit Weight

In many structures, the dead weight of the materials in the structure significantly contributes to the total design stress. If the weight of the materials can be reduced, the size of the structural members can be also reduced. Thus, the weight of the materials is an important design consideration. In addition, in the design of asphalt and concrete mixes, the weight–volume relationship of the aggregates and binders must be used to select the mix proportions.

There are three general terms used to describe the mass, weight, and volume relationship of materials. *Density* is the mass per unit volume of material. *Unit weight* is the weight per unit volume of material. By manipulation of units, it can be shown that

$$\gamma = \rho g \quad (1.12)$$

where

$$\begin{aligned} \gamma &= \text{unit weight} \\ \rho &= \text{density} \\ g &= \text{acceleration of gravity} \end{aligned}$$

Specific gravity is the ratio of the mass of a substance relative to the mass of an equal volume of water at a specified temperature. The density of water is 1 Mg/m^3 in SI units and 62.4 lb/ft^3 in English units at 4°C (39.2°F). According to the definition, specific gravity is equivalent to the density of a material divided by the density of water. Since the density of water in the metric system has a numerical value of 1, the numerical value of density and specific gravity are equal. This fact is often used in the literature where density and specific gravity terms are used interchangeably.

For solid materials, such as metals, the unit weight, density, and specific gravity have definite numerical values. For other materials such as

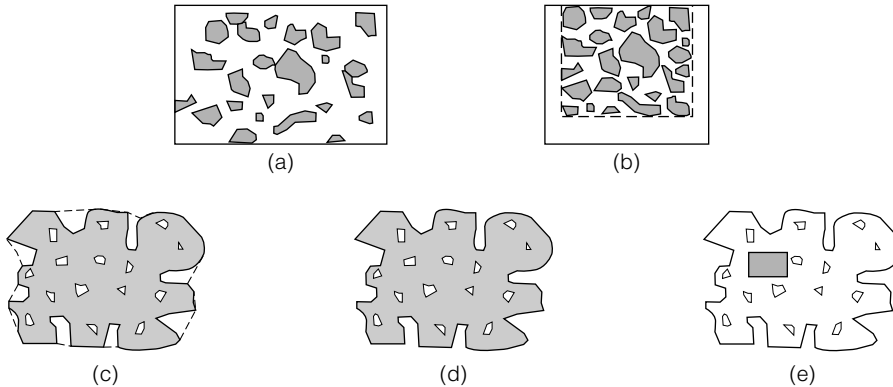


FIGURE 1.16 Definitions of volume used for determining density: (a) loose, (b) compacted, (c) total particle volume, (d) volume not accessible to water, and (e) volume of solids.

wood and aggregates, voids in the materials require definitions for a variety of densities and specific gravities. As shown in Figure 1.16(a) and (b), the bulk volume aggregates will occupy depends on the compaction state of the material. In addition, the density of the material will change depending on how the volume of individual particles is measured. Several types of particle volume can be used, such as the total volume enclosed within the boundaries of the individual particles, volume not accessible to water or asphalt, and volume of solids, as seen in Figure 1.16(c), (d), and (e), respectively. These are important factors in the mix designs of portland cement concrete and asphalt concrete.

1.3.2 ■ Thermal Expansion

Practically all materials expand as temperature increases and contract as temperature falls. The amount of expansion per unit length due to one unit of temperature increase is a material constant and is expressed as the coefficient of thermal expansion

$$\alpha_L = \frac{\delta L / \delta T}{L} \quad (1.13)$$

$$\alpha_V = \frac{\delta V / \delta T}{V} \quad (1.14)$$

where

- α_L = linear coefficient of thermal expansion
- α_V = volumetric coefficient of thermal expansion
- δL = change in the length of the specimen
- δT = change in temperature

- L = original length of the specimen
 δV = change in the volume of the specimen
 V = original volume of the specimen

For isotropic materials, $\alpha_V = 3\alpha_L$.

The coefficient of thermal expansion is very important in the design of structures. Generally, structures are composed of many materials that are bound together. If the coefficients of thermal expansion are different, the materials will strain at different rates. The material with the lesser expansion will restrict the straining of other materials. This constraining effect will cause stresses in the materials that can lead directly to fracture.

Stresses can also be developed as a result of a thermal gradient in the structure. As the temperature outside the structure changes and the temperature inside remains constant, a thermal gradient develops. When the structure is restrained from straining, stress develops in the material. This mechanism has caused brick facades on buildings to fracture and, in some cases, fall off the structure. Also, since concrete pavements are restrained from movement, they may crack in the winter due to a drop in temperature and may “blow up” in the summer due to an increase in temperature. Joints are, therefore, used in buildings, bridges, concrete pavements, and various structures to accommodate this thermal effect.

Sample Problem 1.5

A steel bar with a length of 3 m, diameter of 25 mm, modulus of elasticity of 207 GPa, and linear coefficient of thermal expansion of $0.000009 \text{ m/m}^\circ\text{C}$ is fixed at both ends when the ambient temperature is 40°C . If the ambient temperature is decreased to 15°C , what internal stress will develop due to this temperature change? Is this stress tension or compression? Why?

Solution

If the bar was fixed at one end and free at the other end, the bar would have contracted and no stresses would have developed. In that case, the change in length can be calculated by using Equation 1.13 as follows:

$$\delta L = \alpha_L \times \delta T \times L = 0.000009 \times (-25) \times 3 = -0.000675 \text{ m}$$

$$\varepsilon = \delta L/L = -0.000675/3 = -0.000225 \text{ m/m}$$

Since the bar is fixed at both ends, the length of the bar will not change. Therefore, a tensile stress will develop in the bar as follows:

$$\sigma = \varepsilon E = 0.000225 \times 207000 = 46.575 \text{ MPa}$$

The stress will be tension; in effect, the length of the bar at 15°C without restraint would be 2.999325 m and the stress would be zero. Restraining the bar into a longer condition requires a tensile force.

1.3.3 ■ Surface Characteristics

The surface properties of materials of interest to civil engineers include corrosion and degradation, the ability of the material to resist abrasion and wear, and surface texture.

Corrosion and Degradation Nearly all materials deteriorate over their service lives. The mechanisms contributing to the deterioration of a material differ depending on the characteristics of the material and the environment. Crystalline materials, such as metals and ceramics, deteriorate through a *corrosion* process in which there is a loss of material, either by dissolution or by the formation of nonmetallic scale or film. Polymers, such as asphalt, deteriorate by *degradation* of the material, including the effects of solvent and ultraviolet radiation on the material.

The protection of materials from environmental degradation is an important design concern, especially when the implications of deterioration and degradation on the life and maintenance costs of the structure are considered. The selection of a material should consider both how the material will react with the environment and the cost of preventing the resulting degradation.

Abrasion and Wear Resistance Since most structures in civil engineering are static, the abrasion or wear resistance is of less importance than in other fields of engineering. For example, mechanical engineers must be concerned with the wear of parts in the design of machinery. This is not to say that wear resistance can be totally ignored in civil engineering. Pavements must be designed to resist the wear and polishing from vehicle tires in order to provide adequate skid resistance for braking and turning. Resistance to abrasion and wear is, therefore, an important property of aggregates used in pavements.

Surface Texture The surface texture of some materials and structures is of importance to civil engineers. For example, smooth texture of aggregate particles is needed in portland cement concrete to improve workability during mixing and placing. In contrast, rough texture of aggregate particles is needed in asphalt concrete mixtures to provide a stable pavement layer that resists deformation under the action of load. Also, a certain level of surface texture is needed in the pavement surface to provide adequate friction resistance and prevent skidding of vehicles when the pavement is wet.

1.4 Production and Construction

Even if a material is well suited to a specific application, production and construction considerations may block the selection of the material. Production considerations include the availability of the material and the ability

to fabricate the material into the desired shapes and required specifications. Construction considerations address all the factors that relate to the ability to fabricate and erect the structure on site. One of the primary factors is the availability of a trained work force. For example, in some cities high-strength concrete is used for skyscrapers, whereas in other cities steel is the material of choice. Clearly, either concrete or steel can be used for high-rise buildings. Regional preferences for one material develop as engineers in the region become comfortable and confident in designing with one of the materials and constructors respond with a trained work force and specialized equipment.

1.5 Aesthetic Characteristics

The aesthetic characteristics of a material refer to the appearance of the material. Generally, this characteristic is the responsibility of the architect. However, the civil engineer is responsible for working with the architect to ensure that the aesthetic characteristics of the facility are compatible with the structural requirements. During the construction of many public projects, a certain percentage of the capital budget typically goes toward artistic input. The collaboration between the civil engineer and the architect is greatly encouraged, and the result can increase the value of the structure (see Figure 1.17).

In many cases, the mix of artistic and technical design skills makes the project acceptable to the community. In fact, political views are often more



FIGURE 1.17 An example of artist-engineer collaboration in an engineering project: Air Force Academy, Colorado Springs, Colorado.

difficult to deal with than technical design problems. Thus, engineers should understand that there are many factors beyond the technical needs that must be considered when selecting materials and designing public projects.

1.6 Material Variability

It is essential to understand that engineering materials are inherently variable. For example, steel properties vary depending on chemical composition and method of manufacture. Concrete properties change depending on type and amount of cement, type of aggregate, air content, slump, method of curing, etc. The properties of asphalt concrete vary depending on the binder amount and type, aggregate properties and gradation, amount of compaction, and age. Wood properties vary depending on the tree species, method of cut, and moisture content. Some materials are more homogeneous than others, depending on the nature of the material and the method of manufacturing. For example, the variability of the yield strength of one type of steel is less than the variability of the compressive strength of one batch of concrete. Therefore, variability is an important parameter in defining the quality of civil engineering materials.

When materials from a particular lot are tested, the observed variability is the cumulative effect of three types of variance: the inherent variability of the material, variance caused by the sampling method, and variance associated with the way the tests are conducted. Just as materials have an inherent variability, sampling procedures and test methods can produce variable results. Frequently, statisticians call variance associated with sampling and testing *error*. However, this does not imply the sampling or testing was performed incorrectly. When an incorrect procedure is identified, it is called a *blunder*. The goal of a sampling and testing program is to minimize sampling and testing variance so the true statistical features of the material can be identified.

The concepts of precision and accuracy are fundamental to the understanding of variability. *Precision* refers to the variability of repeat measurements under carefully controlled conditions. *Accuracy* is the conformity of results to the true value or the absence of bias. *Bias* is a tendency of an estimate to deviate in one direction from the true value. In other words, bias is a systematic error between a test value and the true value. A simple analogy to the relationship between precision and accuracy is the target shown in Figure 1.18. When all shots are concentrated at one location away from the center, that indicates good precision and poor accuracy (biased) [Figure 1.18(a)]. When shots are scattered around the center, that indicates poor precision and good accuracy [Figure 1.18(b)]. Finally, good precision and good accuracy are obtained if all shots are concentrated close to the center [Figure 1.18(c)] (Burati and Hughes 1990). Many standardized test methods, such as those of the American Society for Testing and Materials (ASTM) and the American Association of State Highway and Transportation Officials (AASHTO),

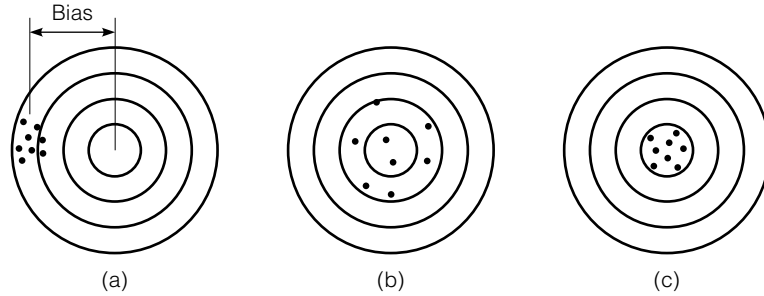


FIGURE 1.18 Exactness of measurements: (a) precise but not accurate, (b) accurate but not precise, and (c) precise and accurate.

contain precision and bias statements. These statements provide the limits of acceptable test results variability. Laboratories are usually required to demonstrate testing competence and can be certified by the American Material Reference Laboratory (AMRL).

1.6.1 ■ Sampling

Typically, *samples* are taken from a *lot* or *population*, since it is not practical or possible to test the entire lot. By testing sufficient samples, it is possible to estimate the properties of the entire lot. In order for the samples to be valid they must be *randomly* selected. Random sampling requires that all elements of the population have an equal chance for selection. Another important concept in sampling is that the sample must be *representative* of the entire lot. For example, when sampling a stockpile of aggregate, it is important to collect samples from the top, middle, and bottom of the pile and to combine them, since different locations within the pile are likely to have different aggregate sizes. The sample size needed to quantify the characteristics of a population depends on the variability of the material properties and the confidence level required in the evaluation.

Statistical parameters describe the material properties. The mean and the standard deviation are two commonly used statistics. The *arithmetic mean* is simply the average of test results of all specimens tested. It is a measure of the central tendency of the population. The *standard deviation* is a measure of the dispersion or spread of the results. The equations for the mean \bar{x} and standard deviation s of a sample are

$$\bar{x} = \frac{\sum_{i=1}^n x_i}{n} \quad (1.15)$$

$$s = \left(\frac{\sum_{i=1}^n (x_i - \bar{x})^2}{n - 1} \right)^{1/2} \quad (1.16)$$

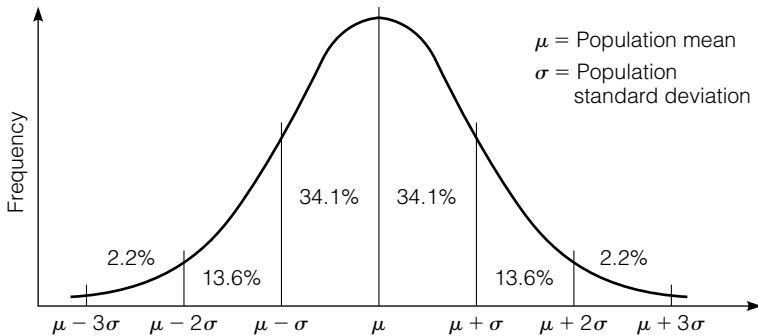


FIGURE 1.19 A normal distribution.

where n is the sample size. The mean and standard deviation of random samples are estimates of the mean and standard deviation, respectively, of the population.

1.6.2 ■ Normal Distribution

The normal distribution is a symmetrical function around the mean, as shown in Figure 1.19. The normal distribution describes many populations that occur in nature, research, and industry, including material properties. The area under the curve between any two values represents the probability of occurrence of an event of interest. Expressing the results in terms of mean and standard deviation, it is possible to determine the probabilities of an occurrence of an event. For example, the probability of occurrence of an event between the mean and ± 1 standard deviation is 68.3%, between the mean and ± 2 standard deviations is 95.5%, and between the mean and ± 3 standard deviations is 99.7%. If a materials engineer tests 20 specimens of concrete and determines the average as 22 MPa and the standard deviation as 3 MPa, the statistics will show that 95.5% of the time the true mean of the population will be in the range of $22 \pm (2 \times 3)$, or 16 to 28 MPa.

1.6.3 ■ Control Charts

Control charts have been used in manufacturing industry and construction applications to verify that a process is in control. It is important to note that control charts do not get or keep a process under control; they provide only a visual warning mechanism to identify when a contractor or material supplier should look for possible problems with the process. Control charts have many benefits (Burati and Hughes 1990), such as

- detect trouble early
- decrease variability
- establish process capability

- reduce price adjustment cost
- decrease inspection frequency
- provide a basis for altering specification limits
- provide a permanent record of quality
- provide basis for acceptance
- instill quality awareness

There are many types of control charts, the simplest of which plots individual results in chronological order. For example, Figure 1.20 shows a control chart of the compressive strength of concrete specimens tested at a ready-mix plant. The control chart can also show the specification tolerance limits so that the operator can identify when the test results are out of the specification requirements. Although this type of control chart is useful, it is based on a sample size of one, and it therefore fails to consider variability within the sample.

Statistical control charts can be developed, such as the control chart for means (X-bar chart) and the control chart for the ranges (R chart) in which the means or the ranges of the test results are chronologically plotted. Figure 1.21(a) shows a control chart for the moving average of each three consecutive compressive strength tests. For example, the first point represents the mean of the first three tests, the second point represents the mean of tests two through four, and so on. Figure 1.21(b) shows a control chart for the moving range of each three consecutive compressive strength tests. The key element in the use of statistical control charts is the proper designation of the *control limits* that are set for a given process. These control limits are not necessarily the same as the tolerance or specification limits and can be set using probability functions. For example, the control chart for means relies on the fact that, for a normal distribution, essentially all of the values fall within ± 3 standard deviations from the mean. Thus, control limits can be set between ± 3 standard deviations from the mean. Warning limits to identify potential problems are sometimes set at ± 2 standard deviations from the mean.

Observing the trend of means or ranges in statistical control charts can help eliminate production problems and reduce variability. Figure 1.22 shows

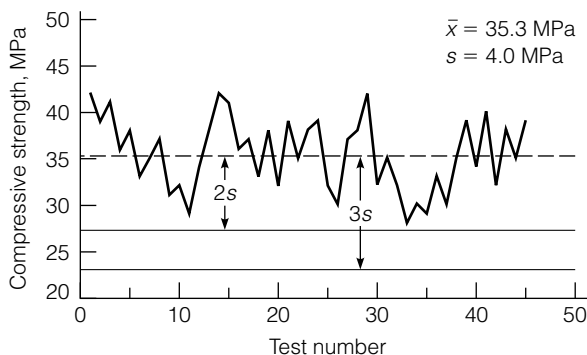


FIGURE 1.20 Control chart of compressive strength of concrete specimens.

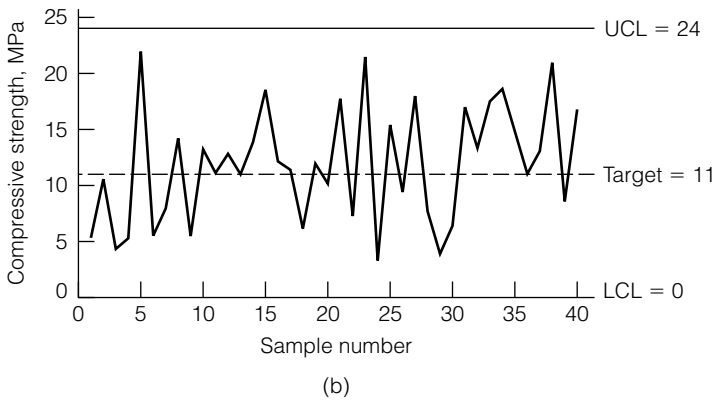
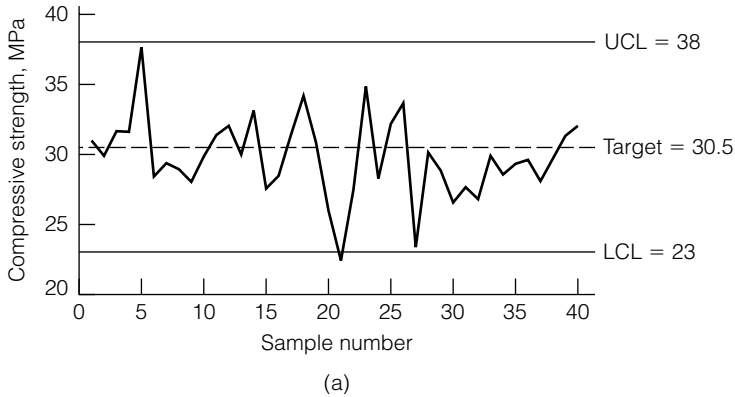


FIGURE 1.21 Statistical control charts: (a) X-bar chart and (b) R chart. UCL indicates upper control limit; LCL indicates lower control limit.

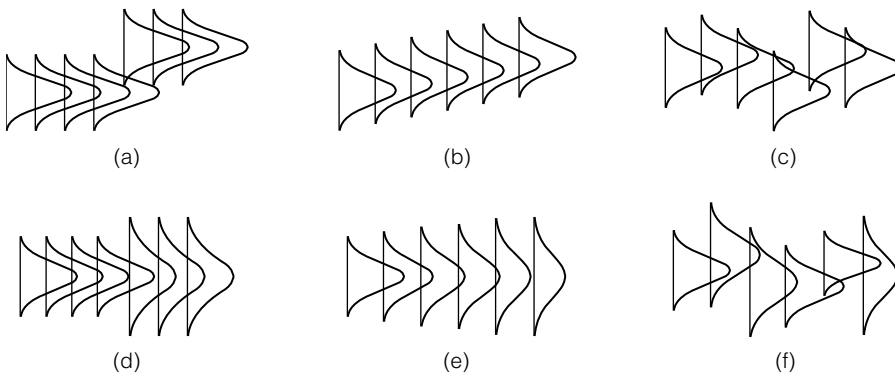


FIGURE 1.22 Possible trends of means and ranges in statistical control charts: (a) sudden change in mean, (b) gradual change in mean, (c) irregular change in mean, (d) sudden change in range, (e) gradual change in range, and (f) irregular change in mean and range.

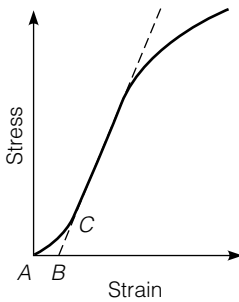


FIGURE 1.23 Correction of toe region in stress–strain curve.

possible trends of means and ranges in statistical control charts (Burati and Hughes 1990). Figure 1.22(a) shows sustained sudden shift in the mean. This could indicate a change of a material supplier during the project. A gradual change in the mean, as illustrated in Figure 1.22(b), could indicate a progressive change brought on by machine wear. An irregular shift in the mean, as shown in Figure 1.22(c), may indicate that the operator is making continuous, but unnecessary, adjustments to the process settings. Figure 1.22(d) shows a sudden change in range, which could also indicate a change of a material supplier during the project. Figure 1.22(e) shows a gradual increase in the range, which may indicate machine wear. Finally, Figure 1.22(f) shows an irregular shift in both mean and range, which indicates a flawed process.

1.6.4 ■ Experimental Error

When specimens are tested in the laboratory, inaccuracy could occur due to machine or human errors. For example, Figure 1.23 shows a stress–strain curve in which a toe region (AC) that does not represent a property of the material exists. This toe region is an artifact caused by taking up slack and alignment or seating of the specimen. In order to obtain correct values of such parameters as modulus, strain, and offset yield point, this artifact must be compensated for in order to give the corrected zero point on the strain axis. This is accomplished by extending the linear portion of the curve backward until it meets the strain axis at point B. In this case, point B is the corrected zero strain point from which all strains must be measured. In the case of a material that does not exhibit any linear region, a similar correction can be made by constructing a tangent to the maximum slope at the inflection point and extending it until it meets the strain axis.

1.7 Laboratory Measuring Devices

Laboratory tests measure material properties. Frequently, specimens are made of the material in question and tested in the laboratory to measure

their response to the applied forces or to certain environmental conditions. These tests require the measurement of certain parameters such as time, deformation, or force. Some of these parameters are measured directly, while others are measured indirectly by relating parameters to each other. Length and deformation can be measured directly using simple devices such as rulers, dial gauges, and calipers. In other cases, indirect measurements are made by measuring electric voltage and relating it to deformation, force, stress, or strain. Examples of such devices include linear variable differential transformers (LVDTs), strain gauges, and load cells. Noncontact deformation measuring devices using lasers and various optical devices are also available. Electronic measuring devices can easily be connected to chart recorders, digital readout devices, or computers, where the results can be easily displayed and processed.

Each device has a certain *sensitivity*, which is the smallest value that can be read on the device's scale. Sensitivity should not be mistaken for accuracy or precision. Magnification can be designed into a gauge to increase its sensitivity, but wear, friction, noise, drift, and other factors may introduce errors that limit the accuracy and precision.

Measurement accuracy cannot exceed the sensitivity of the measuring device. For example, if a stopwatch with a sensitivity of 0.01 second is used to measure time, the smallest time interval that can be recorded is also 0.01 second. The selection of the measuring device and its sensitivity depends on the required accuracy of measurement. The required accuracy, on the other hand, depends on the significance and use of the measurement. For example, when expressing distance of travel from one city to another, an accuracy of 1 kilometer or even 10 kilometers may be meaningful. In contrast, manufacturing a computer microchip may require an accuracy of one-millionth of a meter or better. In engineering tests, the accuracy of measurement must be determined in advance to ensure proper use of such measurements and, at the same time, to avoid unnecessary effort and expense during testing. Many standardized test methods, such as those of ASTM and AASHTO, state the sensitivity of the measuring devices used in a given experiment. In any case, care must be taken to ensure proper calibration, connections, use, and interpretation of the test results of various measuring devices.

Next we will briefly describe measuring devices commonly used in material testing, such as dial gauges, linear variable differential transformers (LVDTs), strain gauges, proving rings, and load cells.

1.7.1 ■ Dial Gauge

Dial gauges are used in many laboratory tests to measure deformation. The dial gauge is attached at two points, between which the relative movement is measured. Most of the dial gauges include two scales with two different pointers, as depicted in Figure 1.24. The smallest division of the large scale determines the sensitivity of the device and is usually recorded on the face of the gauge. One division of the small pointer corresponds to one full rotation of the large pointer. The full range of the small pointer determines the



FIGURE 1.24 Dial gauge.

range of measurement of the dial gauge. Dial gauges used in civil engineering material testing frequently have sensitivities ranging from 0.1 mm to 0.002 mm. The dial gauge shown in Figure 1.24 has a sensitivity of 0.001 in. and a range of 1 in. The gauge can be “zeroed” by rotating the large scale in order to start the reading at the current pointer position.

Dial gauges can be attached to frames or holders with different configurations to measure the deformation of a certain gauge length or the relative movement between two points. For example, the *extensometer* shown in Figure 1.25 is used to measure the deformation of the gauge length of a metal bar during the tensile test. Note that because of the extensometer configuration shown in the figure, the deformation of the bar is one-half of the reading indicated by the dial gauge.

1.7.2 ■ Linear Variable Differential Transformer (LVDT)

The linear variable differential transformer (transducer), or LVDT, is an electronic device commonly used in laboratory experiments to measure small movements or deformations of specimens. The LVDT consists of a nonmagnetic shell and a magnetic core. The shell contains one primary and two secondary electric coils, as illustrated in Figure 1.26. An electric voltage is input to the LVDT and an output voltage is obtained. When the core is in the null position at the center of the shell, the output voltage is zero. When the



FIGURE 1.25 Extensometer with a dial gauge.

core is moved slightly in one direction, an output voltage is obtained. Displacing the core in the opposite direction produces an output voltage with the opposite sign. The relationship between the core position and the output voltage is linear within a certain range determined by the manufacturer. If this relation is known, the displacement can be determined by measuring the output voltage using a voltmeter or a readout device. LVDTs can measure both static and dynamic movements.

LVDTs vary widely in sensitivity and range. The sensitivity of commercially available LVDTs ranges from 0.003 to 0.25 V/mm (0.08 to 6.3 V/in.) of displacement per volt of excitation. Normal excitation supplied to the primary coil is 3 Vac with a frequency ranging from 50 Hz to 10 kHz. If 3 V is used, the most sensitive LVDTs provide an output of 18.9 mV/mm (Dally and

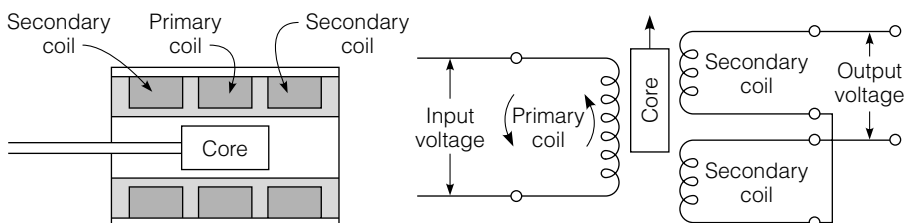


FIGURE 1.26 LVDT circuit.

Riley 1991). In general, very sensitive LVDTs have small linear ranges, whereas LVDTs with greater linear ranges are less sensitive. The sensitivity and the linear range needed depend on the accuracy and the amount of displacement required for the measurement.

Before use, the LVDT must be calibrated to determine the relation between the output voltage and the displacement. A calibration device consisting of a micrometer, a voltmeter, and a holder is used to calibrate the LVDT, as shown in Figure 1.27.

The shell and the core of the LVDT can be either separate or attached in a spring-loaded arrangement (Figure 1.28). When the former type is used, a nonmagnetic threaded connecting rod is attached to the core used to attach the LVDT to the measured object. In either case, to measure the relative movement between two points, the core is attached to one point and the shell is attached to the other point. When the distance between the two points changes, the core position changes relative to the shell, proportionally altering the output voltage. Figure 1.29 shows an extensometer with an

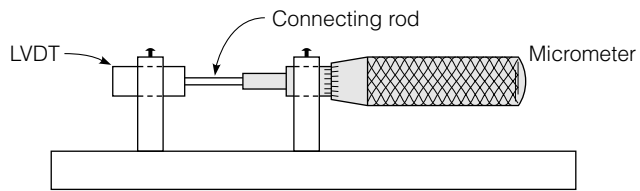


FIGURE 1.27 LVDT calibration device.

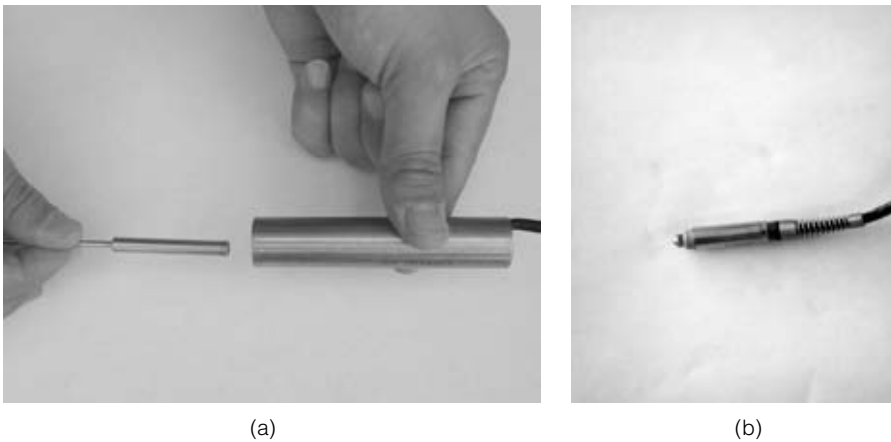


FIGURE 1.28 Types of LVDT: (a) non-spring loaded (core and connecting rod taken out of the shell), and (b) spring loaded (core and connecting rod inside the shell).

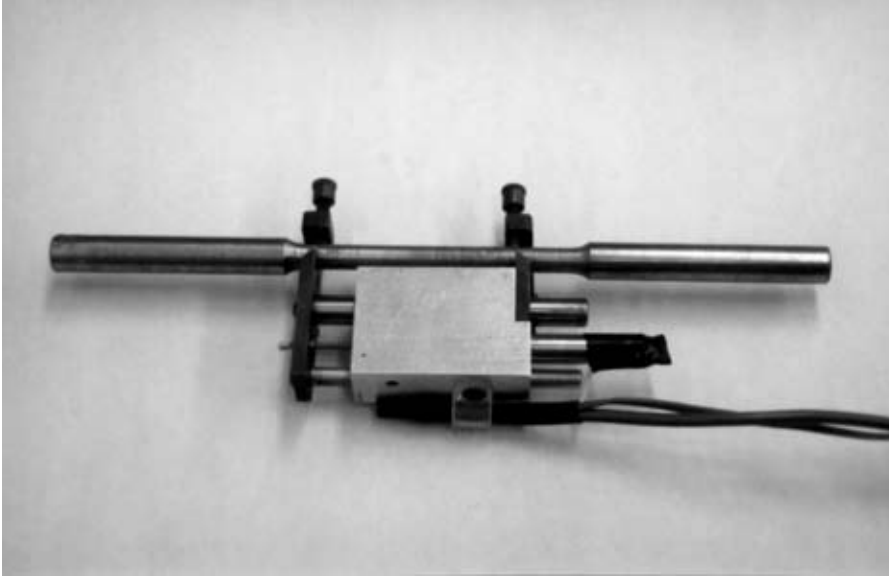


FIGURE 1.29 Extensometer with an LVDT.

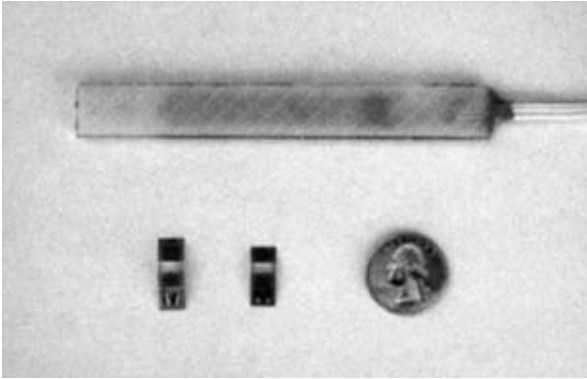
LVDT that can be used to measure the deformation of a metal rod during the tensile test.

1.7.3 ■ Strain Gauge

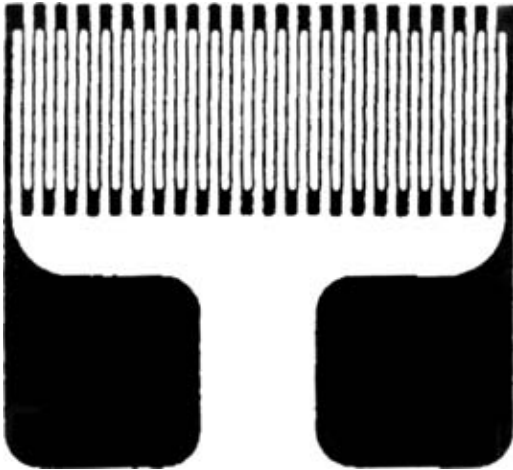
Strain gauges are used to measure small deformations within a certain gauge length. There are several types of strain gauge, but the most dominant type is the electrical strain gauge, which consists of a foil or wire bonded to a thin base of plastic or paper (Figure 1.30). An electric current is passed through the element (foil or wire). As the element is strained, its electrical resistance changes proportionally. The strain gauge is bonded via an adhesive to the surface on which the strain measurement is desired. As the surface deforms, the strain gauge also deforms and, consequently, the resistance changes. Since the amount of resistance change is very small, an ordinary ohmmeter cannot be used. Therefore, special electric circuits, such as the Wheatstone bridge, are used to detect the change in resistance (Dally and Riley 1991).

Strain gauges are manufactured with different sizes, but the most convenient strain gauges have a gauge length of about 5 mm to 15 mm (1/4 in. to 1/2 in.). Larger strain gauges can also be made and used in some applications.

A wire gauge consists of a length of very fine wire (about 0.025 mm diameter) that is looped into a pattern. A foil gauge is made by etching a pattern on a very thin metal foil (about 0.0025 mm thick). Foils or wires are made in a great variety of shapes, sizes, and types and are bonded to a plastic or paper



(a)



(b)

FIGURE 1.30 Strain gauges: (a) strain gauges with different sizes and (b) typical foil strain gauge.

base. When the strain gauge is bonded to the object, it is cemented firmly with the foil or wire side out. Foil–plastic gauges are more commonly used than wire gauges.

When using strain gauges, it is very important to have a tight bond between the gauge and the member. The surface must be carefully cleaned and prepared and the adhesive must be properly applied and cured. The adhesive must be compatible with the material being tested.

1.7.4 ■ Proving Ring

Proving rings are used to measure forces in many laboratory tests. The proving ring consists of a steel ring with a dial gauge attached, as shown in Figure 1.31. When a force is applied on the proving ring, the ring deforms, as measured with the dial gauge. If the relation between the force and dial

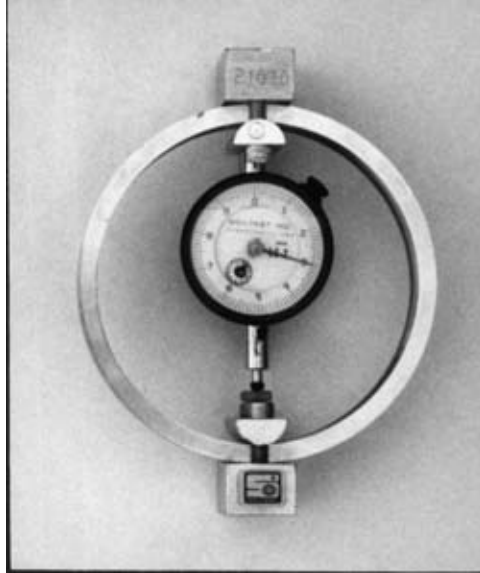


FIGURE 1.31 Proving ring.

gauge reading is known, the proving ring can be used to measure the applied force. Therefore, the proving ring comes with a calibration relationship, either in a form of a linear equation or a table, that allows the user to determine the magnitude of the force based on measuring the deformation of the proving ring. To avoid damage, it is important not to apply a force on the proving ring higher than the capacity specified by the manufacturer. Moreover, periodic calibration of the proving ring is advisable to insure proper measurements.

1.7.5 ■ Load Cell

The load cell is an electronic force-measuring device used for many laboratory tests. In this device, strain gauges are attached to a member within the load cell, which is subjected to either axial loading or bending. An electric voltage is input to the load cell and an output voltage is obtained. If the relation between the force and the output voltage is known, the force can easily be determined by measuring the output voltage.

Load cells are manufactured in different shapes and load capacities. Figure 1.32(a) illustrates a tensile load cell fabricated by mounting four strain gauges on the central region of a tension specimen. Figure 1.32(b) shows an S-shaped load cell in which strain gauges are bonded to the central portion and calibrated to measure the force applied on the top and bottom of the load cell. Figure 1.32(c) illustrates a diaphragm strain gauge bonded to the inside surface of an enclosure that measures the amount of pressure applied on the load cell.

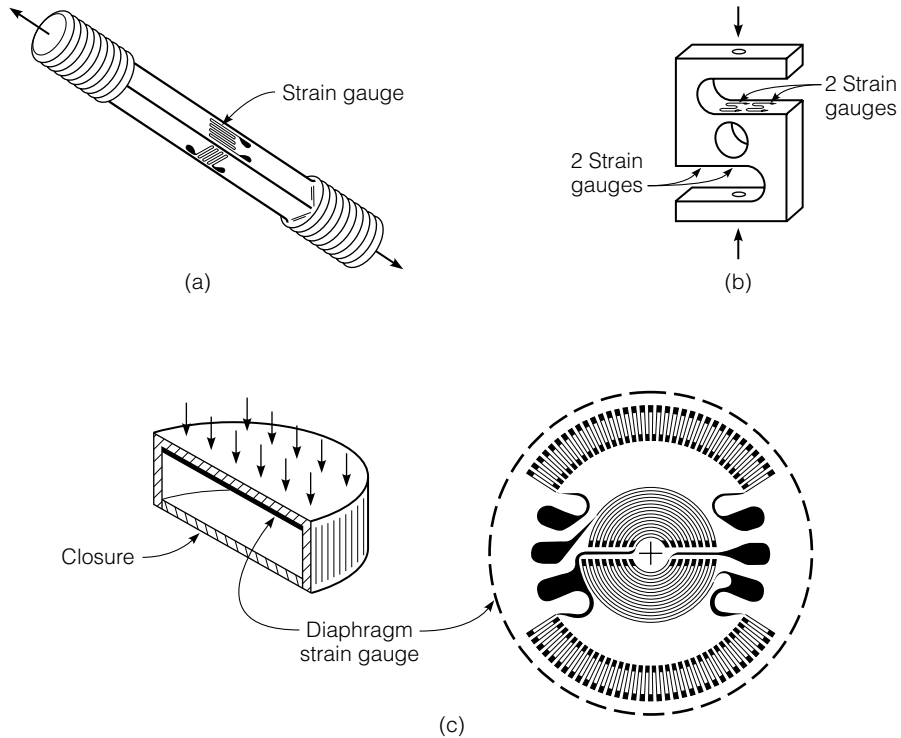


FIGURE 1.32 Load cells: (a) strain gauges on a tension rod, (b) strain gauges on an S-shaped element, and (c) diaphragm strain gauge on a closure.

Load cells must be regularly calibrated using either dead loads or a calibrated loading machine. Care must be taken not to overload the load cell. If the load applied on the load cell exceeds the capacity recommended by the manufacturer, permanent deformation can develop, ruining the load cell.

S U M M A R Y

Civil and construction engineers are involved in the selection of construction materials with the mechanical properties needed for each project. In addition, the selection process must weigh other factors beyond the material's ability to carry loads. Economics, production, construction, maintenance, and aesthetics must all be considered when selecting a material.

Lately, in all fields of engineering, there has been tremendous growth in the use of new high-performance materials. For example, in the automotive

industry, applications of ceramics and plastics are increasing as manufacturers strive for better performance, economy, and safety, while pushing to reduce emissions. Likewise, civil and construction engineers are continuously looking for materials with better quality and higher performance. Advanced composite materials, geotextiles, and various synthetic products are currently competing with traditional civil engineering materials. Although traditional materials such as steel, concrete, wood, and asphalt will continue to be used for some time, improvements of these materials will proceed by changing the molecular structure of such materials and using modifiers to improve their performance. Examples of such improvements include fiber-reinforced concrete, polymer-modified concrete and asphalt, low temperature–susceptible asphalt binder, high-early-strength concrete, superplasticizers, epoxy-coated steel reinforcement, synthetic bar reinforcement, rapid-set concrete patching compounds, prefabricated drainage geocomposites, lightweight aggregates, fire-resistant building materials, and earthquake-resistant joints. Civil engineers are also recycling old materials in an effort to save materials cost, reduce energy, and improve the environment.

Q U E S T I O N S A N D P R O B L E M S

- 1.1 State three examples of a static load application and three examples of a dynamic load application.
- 1.2 A material has the stress–strain behavior shown in Figure P1.2. What is the material strength at rupture? What is the toughness of this material?

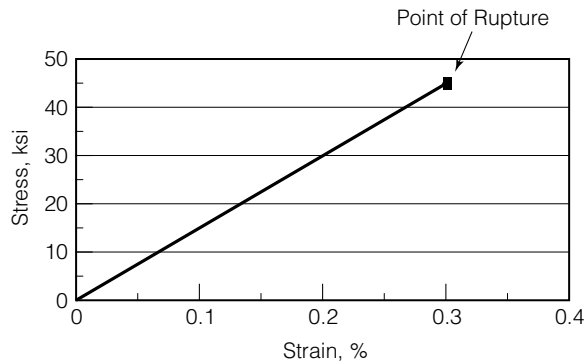


FIGURE P1.2

- 1.3 A tensile load of 220 kN is applied to a round metal bar with a diameter of 16 mm and a gage length of 50 mm. Under this load the bar elastically deforms so that the gage length increases to 50.1686 mm and the diameter decreases to 15.9875 mm. Determine the modulus of elasticity and Poisson's ratio for this metal.
- 1.4 A cylinder with a 6.0 in. diameter and 12.0 in. length is put under a compressive load of 138.4 kips. The modulus of elasticity for this specimen is 6500 ksi and Poisson's ratio is 0.4. Calculate the final length and the final diameter of this specimen under this load.
- 1.5 A rectangular block of aluminum 30 mm × 60 mm × 90 mm is placed in a pressure chamber and subjected to a pressure of 100 MPa. If the modulus of elasticity is 70 GPa and Poisson's ratio is 0.333, what will be the decrease in the longest side of the block, assuming that the material remains within the elastic region? What will be the decrease in the volume of the block?
- 1.6 A material has a stress-strain relationship that can be approximated by the equation

$$\varepsilon = 0.3 \times 10^{-16} \times \sigma^3$$

where the stress is in psi. Find the secant modulus and the tangent modulus for the stress level of 50,000 psi.

- 1.7 On a graph, show the stress-strain relationship under loading and unloading for the following two materials:
- nonlinear elastic material
 - elastoplastic material with strain hardening
- 1.8 The rectangular block shown in Figure P1.8 is subjected to tension within the elastic range. The increase in the length of a is 2×10^{-3} in. and the contraction of b is 3.25×10^{-4} in. If the original lengths of a and b were 2 in. and 1 in., respectively, what is Poisson's ratio for the material of the specimen?

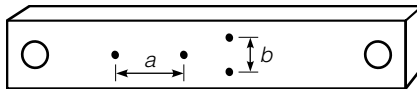


FIGURE P1.8

- 1.9 A cylindrical rod with a length of 380 mm and a diameter of 10 mm is to be subjected to a tensile load. The rod must not experience plastic deformation or an increase in length of more than 0.9 mm when a load of 24.5 kN is applied. Which of the four materials listed in the accompanying table are possible candidates? Justify your answer.

Material	Elastic Modulus, GPa	Yield Strength, MPa	Tensile Strength, MPa
Copper	110	248	289
Aluminum alloy	70	255	420
Steel	207	448	551
Brass alloy	101	345	420

1.10 The stress–strain relation shown in Figure P1.10 was obtained during the tensile test of an aluminum alloy specimen.

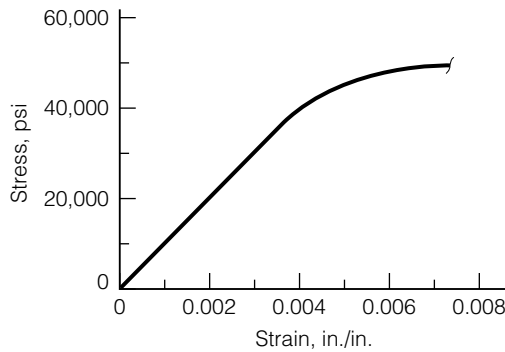


FIGURE P1.10

- Determine the following:
- Young’s modulus within the linear portion
 - Tangent modulus at a stress of 45,000 psi
 - Yield stress using an offset of 0.002 strain
 - If the yield stress in part c is considered failure stress, what is the maximum working stress to be applied to this material if a factor of safety of 1.5 is used?
- 1.11 A tension test performed on a metal specimen to fracture produced the stress–strain relationship shown in Figure P1.11. Graphically determine the following (show units and all work):
- Modulus of elasticity within the linear portion.
 - Yield stress at an offset strain of 0.002 in./in.
 - Yield stress at an extension strain of 0.005 in/in.
 - Secant modulus at a stress of 62 ksi.
 - Tangent modulus at a stress of 65 ksi.

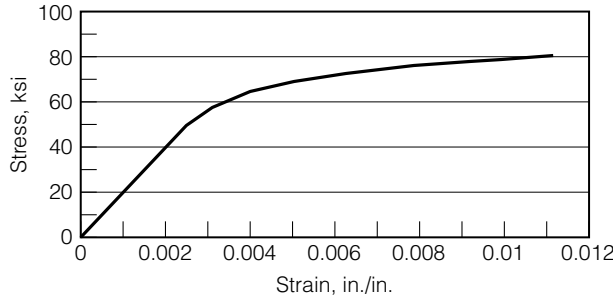


FIGURE P1.11

- 1.12 Use Problem 1.11 to graphically determine the following:
- Modulus of resilience
 - Toughness

Hint: The toughness (u_t) can be determined by calculating the area under the stress–strain curve

$$u_t = \int_0^{\varepsilon_f} \sigma \, d\varepsilon$$

where ε_f is the strain at fracture. The preceding integral can be approximated numerically by using a trapezoidal integration technique:

$$u_t = \sum_{i=1}^n u_i = \sum_{i=1}^n \frac{1}{2} (\sigma_i + \sigma_{i-1}) (\varepsilon_i - \varepsilon_{i-1})$$

- If the specimen is loaded to 40 ksi only and the lateral strain was found to be -0.00057 in./in., what is Poisson's ratio of this metal?
 - If the specimen is loaded to 70 ksi only and then unloaded, what is the permanent strain?
- 1.13 Figure P1.13 shows the stress–strain relations of metals A and B during tension tests until fracture. Determine the following for the two metals (show all calculations and units):
- Proportional limit
 - Yield stress at an offset strain of 0.002 in./in.
 - Ultimate strength
 - Modulus of resilience
 - Toughness
 - Which metal is more ductile? Why?

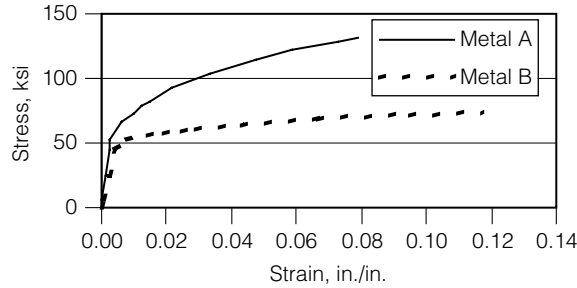


FIGURE P1.13

- 1.14 A brass alloy has a yield strength of 280 MPa, a tensile strength of 390 MPa, and an elastic modulus of 105 GPa. A cylindrical specimen of this alloy 12.7 mm in diameter and 250 mm long is stressed in tension and found to elongate 7.6 mm. On the basis of the information given, is it possible to compute the magnitude of the load that is necessary to produce this change in length? If so, calculate the load. If not, explain why.
- 1.15 Figure P1.15 shows (i) elastic–perfectly plastic and (ii) elastoplastic with strain hardening idealized responses. What stress is needed in each case to have
- a strain of 0.001?
 - a strain of 0.004?

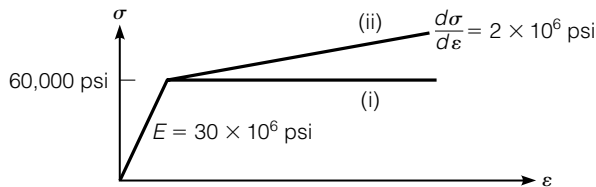


FIGURE P1.15

- 1.16 An elastoplastic material with strain hardening has the stress–strain relation shown in Figure P1.16. The yield point corresponds to 600 MPa stress and 0.003 m/m strain.
- If a bar made of this material is subjected to a stress of 650 MPa and then released, what is the permanent strain?
 - What is the percent increase in yield strength that is gained by the strain hardening shown in part (a)?
 - What is the percent increase in strength that is gained by the strain hardening shown in part (a)?
 - After strain hardening, if the material is subjected to a stress of 625 MPa, how much strain is obtained? Is this strain elastic, permanent, or a combination of both?

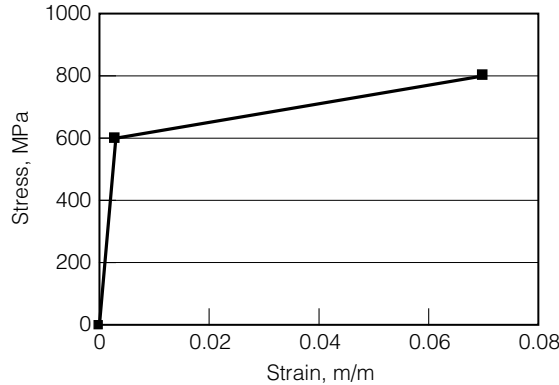


FIGURE P1.16

1.17 An asphalt concrete cylindrical specimen with a 4-inch diameter and a 6-inch height is subjected to an axial compressive static load of 150 lb for 1 hour after which the load is removed. The test was performed at a temperature of 100°F. The height of the specimen was measured at different time intervals during loading and unloading for a total of 2 hours and produced the following results:

Time, minutes	Specimen Height, inches	Time, minutes	Specimen Height, inches
0	6.0000	60.01	5.9772
0.01	5.9916	62	5.9807
2	5.9870	65	5.9841
5	5.9833	70	5.9879
10	5.9796	80	5.9926
20	5.9753	90	5.9942
30	5.9725	100	5.9954
40	5.9708	110	5.9959
50	5.9696	120	5.9964
60	5.9688		

- Using a spreadsheet program, plot the stress versus time and the strain versus time on two separate graphs.
- How much is the elastic strain?
- What is the permanent strain at the end of the experiment?
- What is the phenomenon of the change of specimen height during static loading called? What is the phenomenon of the change of specimen height during unloading called?

- 1.18 Draw a sketch of the Maxwell model and label all components. Draw a graph showing displacement versus time when the model is subjected to a constant force for a time period t and then released. Comment on why the model responds this way.
- 1.19 Derive the response relation for each of the models shown in Figure P1.19, assuming that the force F is constant and instantaneously applied.



FIGURE P1.19

- 1.20 State four failure modes of materials. Describe typical examples of each mode.
- 1.21 A metal rod having a diameter of 10 mm is subjected to a repeated tensile load. The material of the rod has a tensile strength of 290 MPa and a fatigue failure behavior as shown in Figure 1.14. How many load repetitions can be applied to this rod before it fails if the magnitude of the load is (a) 5 kN, and (b) 11 kN?
- 1.22 What is the factor of safety? On what basis is its value selected?
- 1.23 State the typical values of the specific gravity of three materials commonly used in construction.
- 1.24 Define the coefficient of thermal expansion. What is the relation between the linear and the volumetric coefficients of thermal expansion?
- 1.25 A steel rod, which is free to move, has a length of 200 mm and diameter of 20 mm at a temperature of 15°C. If the rod is heated uniformly to 115°C, determine the length and the diameter of this rod to the nearest micron at the new temperature if the coefficient of thermal expansion of steel is 12.5×10^{-6} m/m/°C. Is there a stress on the rod at 115°C?
- 1.26 In Problem 1.25, if the rod is snugly fitted against two immovable nonconducting walls at a temperature of 15°C and then heated uniformly to 115°C, what is its length at 115°C? If the modulus of elasticity of steel is 207 GPa, what is the stress induced in the bar? Is this stress tension or compression?
- 1.27 A 4-meter-long steel plate with a rectangular cross section (10 mm \times 50 mm) is resting on a frictionless surface under the sun. The plate temperature is measured to be at 40°C. The plate is then moved into a cold room and is left to rest on a frictionless surface. After several hours, the plate temperature is measured to be at 5°C.

The steel has a modulus of elasticity equal to 200,000 MPa and a coefficient of thermal expansion equal to 1.1×10^{-5} m/m/°C.

- a. What is the length of shrinkage?
 - b. What tension load is needed to return the length to the original value of 4 meters?
 - c. What is the longitudinal strain under this load?
- 1.28 Estimate the tensile strength required to prevent cracking in a concrete-like material that is cast into a bar, 50 in. long, and is fully restrained at each end against axial movement. The concrete is initially cast and cured at a temperature of 100°F and subsequently cools to a temperature of 0°F. Assume the modulus of elasticity is 5 million psi and the thermal coefficient is 5×10^{-6} in./in./°F.
 - 1.29 State two examples in which corrosion plays an important role in selecting the material to be used in a structure.
 - 1.30 Briefly discuss the variability of construction materials. Define the terms accuracy and precision when tests on materials are performed.
 - 1.31 In order to evaluate the properties of a material, samples are taken and tested. What are the two most important factors that must be satisfied when the sample is collected? Show how these factors can be satisfied.
 - 1.32 A contractor claims that the mean compressive strength for a concrete mix is 32.4 MPa (4700 psi) and that it has a standard deviation of 2.8 MPa (400 psi). If you break 16 cylinders and obtain a mean compressive strength of 30.3 MPa (4400 psi), would you believe the contractor's claim? Why? (*Hint*: Use statistical *t*-test.)
 - 1.33 During construction of the surface layer of asphalt pavement, two samples were taken every day to measure the asphalt content in the mix to ensue that it is within the specification limits. The target value was set at 5.5 percent with a tolerance of ± 0.4 percent. The following asphalt content data were collected:

Date	Asphalt Content, %	Date	Asphalt Content, %
May 5	5.7	May 12	5.3
May 5	5.8	May 12	5.6
May 6	5.3	May 13	5.8
May 6	5.4	May 13	5.1
May 7	5.7	May 14	5.8
May 7	5.7	May 14	5.9
May 8	5.6	May 15	5.1
May 8	5.8	May 15	6.2
May 9	5.4	May 16	5.2
May 9	5.5	May 16	4.8

- a. Calculate the mean, standard deviation, and the coefficient of variation of the data.
 - b. Using a spreadsheet program, create a control chart for these data showing the target value and the upper and lower specification limits. Are the asphalt content data within the specification limits? Comment on any trend and possible reasons.
- 1.34 Briefly discuss the concept behind each of the following measuring devices:
- a. LVDT
 - b. strain gauge
 - c. proving ring
 - d. load cell
- 1.35 Referring to the dial gage shown in Fig. P1.35, determine
- a. Accuracy
 - b. Sensitivity
 - c. Range, given that the small pointer moves one division for each full turn of the large pointer
 - d. Which of the preceding items can be improved through calibration?

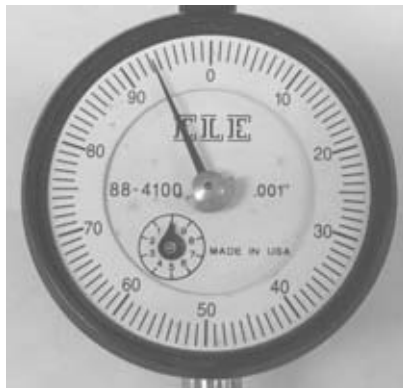


FIGURE P1.35

- 1.36 Measurements should be reported to the nearest 1000, 100, 10, 1, 0.1, 0.01, 0.001 units, etc., depending on their variability and intended use. Using your best judgment, how should the following measurements be reported?
- a. Deformation of a steel specimen during the tension test (in.)
 - b. Tensile strength of steel (psi)
 - c. Modulus of elasticity of aluminum (MPa)
 - d. Weight of aggregate in a sieve analysis test (grams)
 - e. Compressive strength of portland cement concrete (psi)

- f. Moisture content in a concrete masonry unit (percent)
 - g. Asphalt content in hot-mix asphalt (percent)
 - h. Specific gravity of wood
 - i. Distance between New York city and Los Angeles (miles)
 - j. Dimensions of a computer microchip (mm)
- 1.37 During calibration of an LVDT, the data shown in the accompanying table were obtained. Using a spreadsheet program, plot the relation between the micrometer reading and voltage. What is the linear range of the LVDT? Determine the calibration factor of the LVDT by obtaining the best fit line of the data within the linear range.

Micrometer Reading, inches	Voltage, Volts	Micrometer Reading, inches	Voltage, Volts
0.342	-10.120	0.472	0.985
0.352	-10.120	0.482	1.990
0.362	-10.121	0.492	3.023
0.372	-9.134	0.502	4.035
0.382	-8.131	0.512	5.071
0.392	-7.111	0.522	6.098
0.402	-6.100	0.532	7.115
0.412	-5.108	0.542	8.143
0.422	-4.089	0.552	9.144
0.432	-3.097	0.562	10.157
0.442	-2.059	0.572	10.156
0.452	-1.053	0.582	10.156
0.462	0		

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2

NATURE OF MATERIALS

To a large extent, the behavior of materials is dictated by the structure and bonding of the atoms that are the building blocks for all matter. Knowledge of the bonding and structure of materials at the molecular level allows us to understand their behavior. The chapter presents a broad overview of concepts essential to our understanding of these behaviors. The chapter reviews the basic types of bonds and then, based on the type of bond, classifies materials as metallic, ceramic, or amorphous. The general nature of each of these classes of materials is presented.

2.1 Basic Materials Concepts

Atoms are the basic building blocks of all materials. For the purpose of this text, atoms will be considered to consist of three subatomic particles: *protons*, *neutrons*, and *electrons*. The protons and neutrons are at the center of the atom, while the electrons travel about the nucleus in paths or shells. The *atomic number* is the number of protons in the nucleus of the atom. The *atomic mass* of an atom is the number of protons plus the number of neutrons in the center of the atom. An *element* is an atom or group of atoms with the same atomic number. *Isotopes* are elements with different numbers of neutrons in the nucleus.

2.1.1 ■ Electron Configuration

The behavior of an atom's electrons controls, to a large extent, the characteristics of an element. An electrically neutral (or complete) atom has equal numbers of electrons and protons. However, an atom may either release or attract electrons to reach a more stable configuration. Electrons travel around the nucleus in orbital paths, or orbits, around the nucleus. The distance between electrons and the nucleus is not fixed; it is better described as a random variable with a distribution, as shown in Figure 2.1.

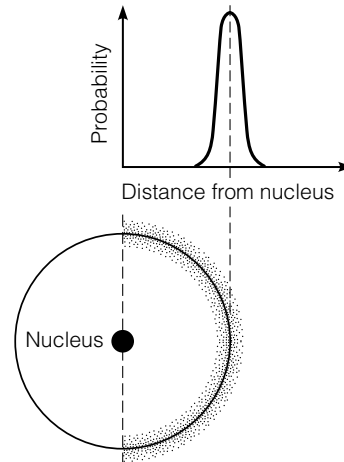


FIGURE 2.1 Orbital path of model electrons.

Each orbital path, or *shell*, can hold only a fixed number of electrons. The maximum number of electrons in a shell is equal to $2n^2$, where n is the principal quantum number of the shell. The orbital path of electrons is defined by four parameters: the principal quantum number or shell designation, the subshell designation, the number of energy states of the subshell, and the spin of the electron. Table 2.1 lists the number of electrons that can be in each shell and subshell.

The energy level of an electron depends on its shell and subshell location, as shown in Figure 2.2 (Callister 2003). Electrons always try to fill the lowest energy location first. As shown in Figure 2.2, for a given subshell, the larger the principal quantum number, the higher is the energy level

TABLE 2.1 Number of Available Electron States in Some of the Electron Shells and Subshells

Principal Quantum Number	Shell Designation	Subshell	Number of States	Number of Electrons	
				per subshell	per shell
1	K	s	1	2	2
2	L	s	1	2	8
		p	3	6	
3	M	s	1	2	18
		p	3	6	
		d	5	10	
4	N	s	1	2	32
		p	3	6	
		d	5	10	
		f	7	14	

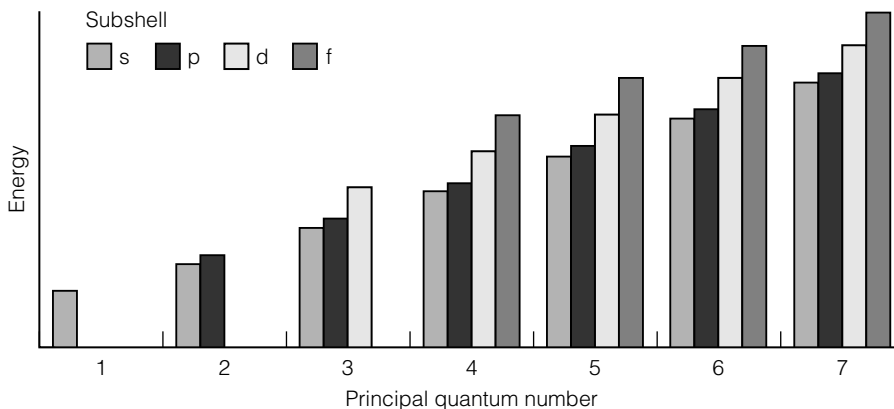


FIGURE 2.2 Energy levels of electrons in different shells and subshells.

(e.g., the energy of subshell 1s is less than subshell 2s). Within a shell, the energy level increases with the subshell location: Subshell f is a higher energy state than subshell d, and so on. There is an energy overlap between the shells: Subshell 4s is at a lower energy state than subshell 3d. The electron configuration, or structure of an atom, describes the manner in which the electrons are located in the shells and subshells. The convention for listing the location of shells in the atom is to list the quantum number, followed by the subshell designation, followed by the number of electrons in the subshell raised to a superscript. This sequence is repeated for each subshell that contains electrons. Table 2.2 provides some examples.

TABLE 2.2 Sample Electron Configurations

Element	Atomic Number	Electron Configuration
Hydrogen	1	$1s^1$
Helium	2	$1s^2$
Carbon	6	$1s^2 2s^2 2p^2$
Neon	10	$1s^2 2s^2 2p^6$
Sodium	11	$1s^2 2s^2 2p^6 3s^1$
Aluminum	13	$1s^2 2s^2 2p^6 3s^2 3p^1$
Silicon	14	$1s^2 2s^2 2p^6 3s^2 3p^2$
Sulfur	16	$1s^2 2s^2 2p^6 3s^2 3p^4$
Argon	18	$1s^2 2s^2 2p^6 3s^2 3p^6$
Calcium	20	$1s^2 2s^2 2p^6 3s^2 3p^6 4s^2$
Chromium	24	$1s^2 2s^2 2p^6 3s^2 3p^6 3d^5 4s^1$
Iron	26	$1s^2 2s^2 2p^6 3s^2 3p^6 3d^6 4s^2$
Copper	29	$1s^2 2s^2 2p^6 3s^2 3p^6 3d^{10} 4s^1$

The electrons in the outermost filled shell are the *valence electrons*. These are the electrons that participate in the formation of primary bonds between atoms. The eight electrons needed to fill the s and p subshells are particularly important. If these subshells are completely filled, the atoms are very stable and virtually nonreactive, as is the case for the noble gases neon and argon. In many cases atoms will release, attract, or share electrons to complete these subshells and reach a stable configuration. Calcium and chromium are examples of atoms with electrons that fill the 4s subshell, while the 3d subshell is incomplete or empty, as would be expected from Figure 2.2. Copper demonstrates that there are exceptions to the energy rule. One would expect copper to have two electrons in the 4s subshell, leaving nine electrons for the 3d subshell. However, it has ten electrons in the 3d subshell and only one in the 4s subshell. A similar disparity exists for chromium. Note that iron has two electrons in the 4s subshell; thus, it has two electrons more than it needs for a stable configuration. These two electrons are readily released to form iron molecules. Aluminum is also an exception, since it has an excess of three electrons.

2.1.2 ■ Bonding

As two atoms are brought together, both attractive and repulsive forces develop. The effects of these forces are additive, as shown in Figure 2.3, such that once the atoms are close enough to interact, they will reach a point at which the attractive and repulsive forces are balanced and an equilibrium is reached. Energy is required either to bring the atoms closer together

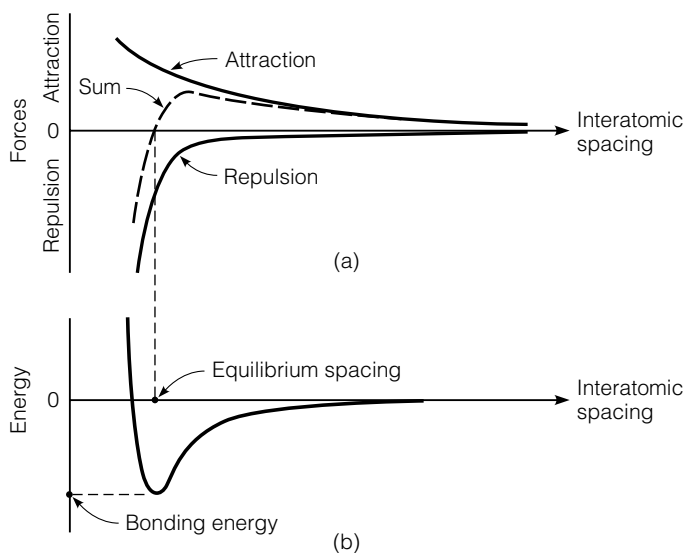


FIGURE 2.3 Attractive and repulsive (a) forces, and (b) energies between atoms. (© Pearson Education, Inc. Used by permission.)

(*compression*) or to separate them (*tension*). The distance at which the net force is zero corresponds to the minimum energy level and is called the *equilibrium spacing*. The minimum energy is represented by a negative sign. The largest negative value is defined as the bonding energy. The bonding energy can be computed from equations for the attractive and repulsive forces. Based on the strength of the bonds, the theoretical strength of a material can be estimated. However, this theoretical strength grossly overestimates the actual strength due to flaws in the molecular structure (Van Vlack 1964, 1989).

The bonding energy depends on the molecular mechanism holding the atoms together. There are two basic categories of bonds: primary and secondary. *Primary bonds* form when atoms interact to change the number of electrons in their outer shells so as to achieve a stable and nonreactive electron structure similar to that of a noble gas. *Secondary bonds* are formed when the physical arrangement of the atoms in the molecule results in an imbalanced electric charge; one side is positive and the other is negative. The molecules are then bonded together through electrostatic force.

Primary Bonds Three types of primary bonds are defined, based on the manner in which the valence electrons interact with other atoms:

1. *ionic bonds*—transfer of electrons from one elemental atom to another (Figure 2.4)
2. *covalent bonds*—sharing of electrons between specific atoms (Figure 2.5)
3. *metallic bonds*—mass sharing of electrons among several atoms (Figure 2.6)

Ionic bonds are the result of one atom releasing electrons to other atoms that accept the electrons. Each of the elements reaches a stable electron configuration of the outer s and p subshells. All of the atoms are ions, since they have an electrical charge. When an atom releases an electron, the atom becomes positively charged; the atom receiving the electron becomes negatively charged.

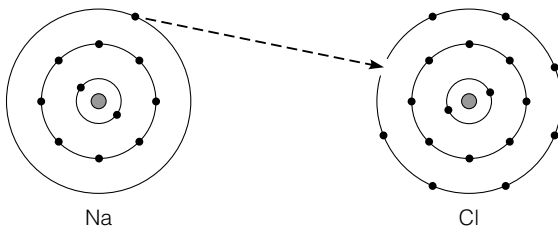


FIGURE 2.4 Ionic bonding.
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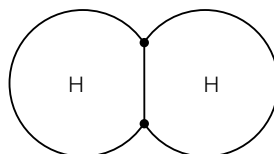


FIGURE 2.5 Covalent bonding.
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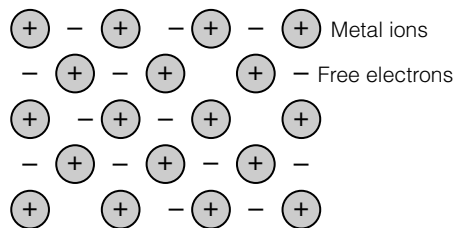


FIGURE 2.6 Metallic bonding. (© Pearson Education, Inc. Used by permission.)

charged. An ion with a positive charge is a *cation* and one with a negative charge is an *anion*. The ionic bond results from the electrostatic attraction of the negatively and positively charged atoms. Since these bonds are based on the transfer of electrons, they have no directional nature.

Covalent bonds occur when two similar atoms *share* electrons in the outer subshell. The atomic orbitals of the atoms overlap and an electron in each atom can exchange with an electron in its partner atom. If the shared electron is considered to be attached to either of the atoms, that atom would have the s and p subshells filled and would therefore be a stable atom. Since the orbital paths of the atoms must overlap for the covalent bond to form, these bonds are highly directional. In materials such as diamond, the covalent bonds are very strong. However, the carbon chain structure of polymers is also formed by covalent bonds and these elements display a wide range of bond energy. The number of bonds that form depends on the number of valence electrons. An atom with N electrons in the valence shell can bond with only $8N$ neighbors by sharing electrons with them. When the number of electrons N equals 7, the atoms join in pairs. When N equals 6, as in sulfur, long chains can form since the atom can bond with two neighbors. When N equals 5, a layered structure can be developed. If there are 4 valence electrons, three-dimensional covalent bonds can result (e.g., the structure of carbon in diamonds) (Jastrzebski 1987). The calcium-silicate chain in portland cement concrete is based on covalent bonds.

Most interatomic bonds are partially ionic and partially covalent, and few compounds have pure covalent or ionic bonds. In general, the degree of either type of bonding depends on the relative positions of the elements in the periodic table. The wider the separation, the more ionic bonds form. Since the electrons in ionic and covalent bonds are fixed to specific atoms, these materials have good thermal and electrical insulation properties.

Metallic bonds are the result of the metallic atoms having loosely held electrons in the outer s subshell. When similar metallic atoms interact, the outer electrons are released and are free to float between the atoms. Thus, the atoms are ions that are electrically balanced by the free electrons. In other words, the free electrons disassociate themselves from the original atom and do not get attached to another atom. Metallic bonds are not directional, and the spacing of the ions depends on the physical characteristics of the atoms. The atoms tend to pack together to give simple, high density structures, like marbles shaken down in a box. The easy movement

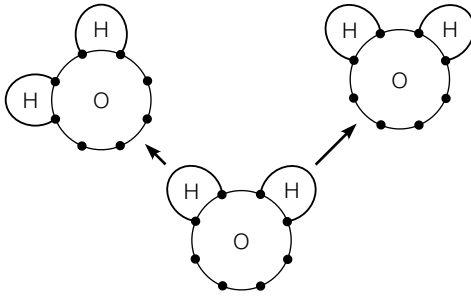


FIGURE 2.7 Secondary bond: hydrogen bridge. (© Pearson Education, Inc. Used by permission.)

of the electrons leads to the high electrical and thermal conductivity of metals.

Secondary Bonds Secondary bonds are much weaker than primary bonds, but they are important in the formation of links between polymer materials. These bonds result from a dipole attraction between uncharged atoms. As the electrons move about the nucleus, at any instant the charge is distributed in a nonsymmetrical manner relative to the nucleus. Thus, at any instant one side of the atom has a negative charge and the other side of the atom has a positive charge. Secondary bonds result from electrostatic attraction of the dipoles of the atoms. These dipole interactions occur between induced dipoles or polar molecules that have permanent dipoles; both types of interactions are classified as *van der Waals forces*. Hydrogen, however, has only one proton and one electron; thus, it tends to form a polar molecule when bonded with other atoms. The electrostatic bond formed due to the hydrogen bond is generally stronger than van der Waals forces, so the hydrogen bond, shown in Figure 2.7, is a special form of the secondary bond.

2.1.3 ■ Material Classification by Bond Type

Based on the predominant type of bond a material's atoms may form, materials are generally classified as metal, inorganic solids, and organic solids. These materials have predominantly metallic, covalent and ionic, and covalent bonds, respectively. Solids with each of these types of bonds have distinctly different characteristics. Metals and inorganic solids generally have a *crystalline* structure, a repeated pattern or arrangement of the atoms. On the other hand, organic solids usually have a random molecular structure. The following are the predominant materials used in civil engineering in each category:

Metallic

steel

iron

aluminum

Inorganic solids

portland cement concrete

bricks and cinder blocks

- glass
- aggregates (rock products)
- Organic solids
 - asphalt
 - plastics
 - wood

2.2 Metallic Materials

The chemical definition of a metal is an element with one, two, or three valence electrons. These elements bond into a mass with metallic bonds. Due to the nature of metallic bonds, metals have a very regular and well-defined structure. Since the metallic bonds are nondirectional, the atoms are free to pack into a dense configuration. The regular three-dimensional geometric pattern of the atoms in a metal is called a *unit cell*. Repeated coalescing of unit cells forms a space lattice of the material. However, in a mass of material, a perfect structure can be achieved only through carefully controlled conditions. Generally, metallic solids are formed by cooling a mass of molten material. As the material cools, the atoms are vibrating. This can cause one atom to occupy the space of two atoms, generating a defect in the lattice structure. In addition, during the cooling process crystals grow simultaneously from several nuclei. As the material continues to cool, these crystals grow together with a boundary forming between the grains. This produces flaws or slip planes in the structure that have an important influence on the behavior and characteristics of the material. In addition, rarely are pure elemental metals used for engineering applications. Even highly refined materials contain impurities that were not removed in the refining process. In addition, most metals do not have desirable properties in a pure state. For example, iron and aluminum used for structural applications have alloying elements that impart special characteristics to the metal. As a result, understanding the nature of metals at the molecular level requires an examination of the primary structure of the metal, the effect of cooling rates, and the impact of impurities and alloying elements.

2.2.1 Lattice Structure

As metals cool from the molten phase, the atoms are arranged into definite structures dependent on the size of the atom and the valence electrons. Certain characteristics are apparent in the three-dimensional array of points formed by the intersection of the parallel lines shown in Figure 2.8. In this configuration, the arrangement of neighboring points about any specific point is identical with the arrangement around any other internal point. This property can be described mathematically by three unit vectors, **a**, **b**,

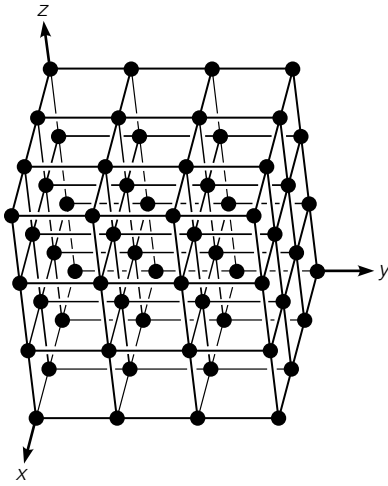


FIGURE 2.8 Parallelogram structure for crystal lattices. (© Pearson Education, Inc. Used by permission.)

and **c**. The location of any point, \mathbf{r}' , relative to a reference point, can be defined in terms of an integer number of vector movements:

$$\mathbf{r}' = \mathbf{r} + n_1\mathbf{a} + n_2\mathbf{b} + n_3\mathbf{c} \quad (2.1)$$

A continuous repetition of Equation 2.1, using incremental integers for n_1 , n_2 , and n_3 will result in the parallelogram shown in Figure 2.8, where \mathbf{r} is the position vector of the origin relative to the reference point. It should be noted that the angles between the axes need not be 90 degrees. Such a network of lines is called a *space lattice*. There are 14 possible space lattices in three dimensions that can be described by vectors \mathbf{a} , \mathbf{b} , and \mathbf{c} . However, the space lattices of common engineering metals can be described by two cubic structures and one hexagonal structure, as shown in Figure 2.9.

A simple cubic lattice structure has one atom on each corner of a cube that has axes at 90 degrees and equal vector lengths. However, this is not a common structure, although it does exist in some metals. There are two important variations on the cubic structure: the *face center cubic* and the *body center cubic*. The face center cubic (FCC) structure has an atom at each corner of the cube plus an atom on each of the faces, as shown in Figure 2.9(a). The body center cubic (BCC) structure has one atom on each of the corners plus one in the center of the cube, as shown in Figure 2.9(b).

The third common metal lattice structure is the *hexagonal close pack* (HCP). As shown in Figure 2.9(c), the HCP has top and bottom layers, with atoms at each of the corners of the hexagon and one atom in the center of the top and bottom planes; in addition, there are three atoms in a center plane. The atoms in the center plane are equidistant from all neighboring atoms. See Table 2.3 for the crystal structures and the atomic radii of some metals.

Two of the important characteristics of the crystalline structure are the coordination number and the atomic packing factor. The *coordination number* is the number of “nearest neighbors.” The coordination number is 12 for FCC

In order to compute the atomic packing factor, the *equivalent number of atoms* associated with each unit cell must be determined, along with the atomic radius of the atoms, which are given in Table 2.3. The equivalent number of atoms associated with a cell is the number of atoms in a large block of material divided by the number of unit cells in the block. However, by properly considering the fraction of atoms in Figure 2.9, we can count the number of “whole” atoms in each unit cell. The FCC structure has eight corner atoms, each of which is shared with seven other unit cells; thus, all the corner atoms contribute one atom to the atom count for the unit cell. The face atoms are shared only with one other unit cell so that each of the face atoms contributes one-half atom. Since there are six faces, the face atoms add 3 atoms to the count. Adding the face and corner atoms yields a total of four atoms. The BCC structure has the equivalent of only two atoms. The HCP structure has an equivalent of 6 atoms. Each of the 12 corner atoms is shared between six unit cells, the face atoms are shared between two unit cells, and the three atoms on the center planes are not shared with any other unit cells, so $12/6 + 2/2 + 3 = 6$.

The volume of the atoms in the unit cell can then be computed as the volume of one atom times the number of equivalent atoms. The volume of a sphere is $V = (4/3)\pi r^3$, where r is the radius. The volume of the unit cell can be determined by knowing the radius of the atoms and the fact that the atoms are in contact with each other.

Sample Problem 2.1

Show that the atomic packing factor for the FCC lattice structure (Fig. SP2.1) is 0.74.



FIGURE SP2.1

Solution

Number of equivalent whole atoms in each unit cell = 4

$$\text{Volume of the sphere} = \left(\frac{4}{3}\right)\pi r^3$$

$$\text{Volume of atoms in the unit cell} = 4 \times \left(\frac{4}{3}\right)\pi r^3 = \left(\frac{16}{3}\right)\pi r^3$$

By inspection, the diagonal of the face of an FCC unit cell = $4r$

Length of each side of unit cell = $2\sqrt{2}r$

Volume of unit cell = $(2\sqrt{2}r)^3$

$$\text{APF} = \frac{\text{volume of atoms in the unit cell}}{\text{total unit volume of the cell}} = \frac{\left(\frac{16}{3}\right)\pi r^3}{(2\sqrt{2}r)^3} = 0.74$$

Similar geometric considerations allow for the calculation of the atomic packing factor for the BCC and HCP structures as 0.68 and 0.74, respectively.

The density of a metal is a function of the type of lattice structure and can be determined from

$$r = \frac{nA}{V_c N_A} \quad (2.3)$$

where

r = density of the material

n = number of equivalent atoms in the unit cell

A = atomic mass of the element (grams/mole)

V_c = volume of the unit cell

N_A = Avogadro's number (6.023×10^{23} atoms/mole)

Sample Problem 2.2

Calculate the radius of the aluminum atom, given that aluminum has an FCC crystal structure, a density of 2.70 Mg/m^3 , and an atomic mass of 26.98 g/mole . Note that the APF for the FCC lattice structure is 0.74.

Solution

$$r = \frac{nA}{V_c N_A}$$

For an FCC lattice structure, $n = 4$, so

$$V_c = \frac{4 \times 26.98}{2.70 \times 10^6 \times 6.023 \times 10^{23}} = 6.636 \times 10^{-29} \text{ m}^3$$

$$\text{APF} = 0.74 = \frac{4 \times \left(\frac{4}{3}\right)\pi r^3}{6.636 \times 10^{-29}}$$

$$r^3 = 0.293 \times 10^{-29}$$

$$r = 0.143 \times 10^{-9} \text{ m} = 0.143 \text{ nm}$$

Knowledge of the type of lattice structure is important when determining the mechanical behavior of a metal. Under elastic behavior, the bonds of the atoms are stretched, but when the load is removed, the atoms return to their original position. On the other hand, plastic deformation, by definition, is a permanent distortion of the materials; therefore, plastic deformation must be associated with a change in the atomic arrangement of the metal. Plastic deformation is the result of planes of atoms slipping over each other due to the action of shear stress. Naturally, the slip will occur on the planes that are the most susceptible to distortion. Since the basic bonding mechanism of the various metals is similar, differences in the theoretical strength of a material are attributed to the differences in the number and orientation of the slip planes that result from the different lattice structures.

2.2.2 ■ Lattice Defects

Even under special circumstances, it is very difficult to grow perfect crystalline structures. Generally, pure crystalline structures are limited to 1 micron in diameter. These pure materials have a strength and modulus of elasticity approaching the maximum theoretical values based on the bonding characteristics. However, the strength and deformation of all practical materials are limited by defects. There are several causes for the development of defects in the crystal structure. These can be classified as follows:

1. point defects or missing atoms
2. line defects or rows of missing atoms, commonly called an *edge dislocation*
3. area defects or grain boundaries
4. volume defects or cavities in the material

In the case of point defects, single atoms can be missing in the lattice structure because the atoms are vibrating as they transition from liquid to solid. As a result, one atom may vibrate in the area where two atoms should be in the lattice. Vacancies have little effect on the properties of the material.

In considering the differences between manufactured and theoretical material behavior, an understanding of line defects becomes important. A typical line defect is shown in Figure 2.10, where a line of missing atoms extends back into the illustration (Van Vlack 1989; Guy and Hren 1974). The atoms above the dislocation are in compression and those below the

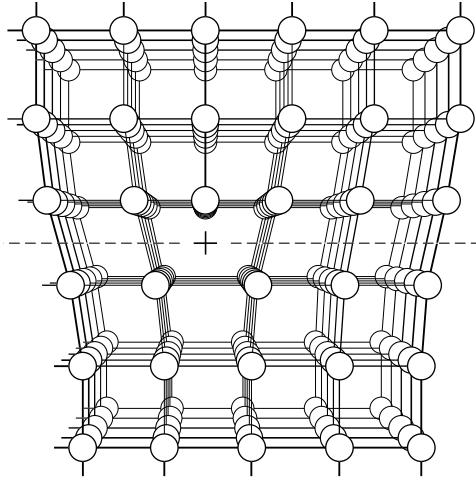


FIGURE 2.10 Atomic packing at a line defect. (© Pearson Education, Inc. Used by permission.)

dislocation are in tension. As a result, the atoms are not at their natural spacing, so the bonds of these atoms are not at the point of minimum energy, as shown in Figure 2.2. Thus, when a shear stress is applied to this location, there will be a tendency for the atoms to slip in a progressive manner from position (a) to (b) to (c), as shown in Figure 2.11 (Flinn and Trojan 1986; Budinski 1996).

Volume defects are flaws in the manufactured material; they will not be discussed further. Area defects are discussed next.

2.2.3 ■ Grain Structure

The structure of metals has been described in terms of the unit cell or the repeated crystalline structure. However, equally important to the behavior of

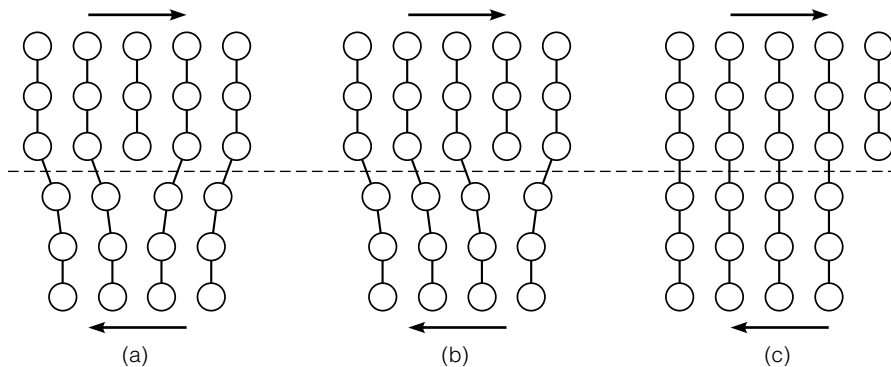


FIGURE 2.11 Plastic deformation involving movement of atoms along a slip plane.

the material is the size and arrangement of the grains in the material. The grain structure (microscopic structure) of the material should be distinguished from the atomic structure. Figure 2.12, for example, is an optical photomicrograph of a low-carbon steel. This photomicrograph was obtained using a scanning electron microscope with a magnification of 500 times. Note that this microscopic scale is much different than the atomic scale of Figure 2.9, which has a magnification in the order of 10,000,000 times.

In metals manufacturing, the material is heated to a liquid, impurities are removed from the stock material, and alloying agents are added. An *alloy* is simply the addition of a second element to a metal. As the material cools from the liquid state, crystals form. Under normal cooling conditions, multiple nuclei will form, producing multiple crystals. As these crystals grow, they will eventually contact each other, forming boundaries. For a given material, the size of the grains depends primarily on the rate of cooling. Under rapid cooling, multiple nuclei are formed, resulting in small grains with extensive boundaries.

There are four types of grain boundaries: coherent, coherent strain, semicoherent, and incoherent, as shown in Figure 2.13. At the coherent

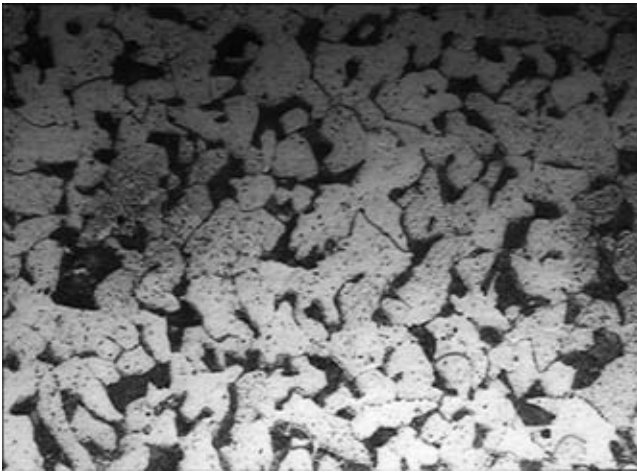


FIGURE 2.12 Optical photomicrograph of low-carbon steel (magnification: 500x).

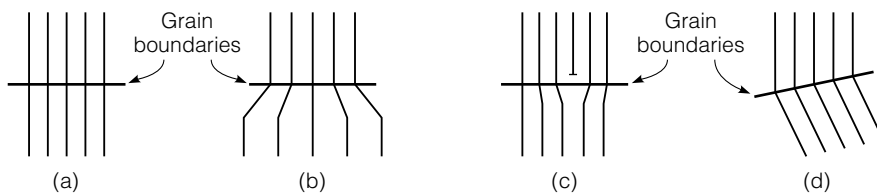


FIGURE 2.13 Types of grain boundaries: (a) coherent, (b) coherent strain, (c) semi-coherent, and (d) incoherent.

boundary, the lattices of the two grains align perfectly, and, in essence, there is no physical boundary. Coherent strain boundaries have a different spacing of atoms on each side of the boundary, which may be the effect of an alloying agent. As with the line dislocation, there will be a strain in the atoms on each side of the boundary due to the difference in the spacing of the atoms. The semicoherent boundary has a different number of atoms on each side of the boundary; therefore, not all of the boundaries can match up. Again, this is similar to line dislocation, with strain occurring on each side of the boundary. The incoherent boundary has a different orientation of the crystals on each side of the boundary; thus, the atoms do not match up in a natural manner.

Grain boundaries have an important effect on the behavior of a material. Although the bonds across the grain boundary do not have the strength of the pure crystal structure, the grain boundaries are at a higher energy state than the atoms away from the grain boundary. As a result, when slip occurs along one of the slip planes in the crystal, it is blocked from crossing the grain boundary, and can be diverted to run along the grain boundary. This increases the length of the slip path, thus requiring more energy to deform or fracture the material. Therefore, reducing grain size increases the strength of a material.

The grain structure of metals is affected by plastic strains. Fabrication methods frequently involve plastic straining to produce a desired shape (e.g., wire is produced by forcing a metal through successively smaller dies to reduce the dimension of the metal from the shape produced in the mill to the desired wire diameter.)

Heat treatments are used to refine grain structure. There are two basic heat treatment methods: *annealing* and *hardening*. Both processes involve heating the material to a point at which the existing grain boundaries will break down and re-form upon cooling. The differences between the processes include the temperature to which the material is heated, the amount of time it is held at the elevated temperature, and the rate of cooling. In hardening, rapid cooling is achieved by immersing the material in a liquid. In annealing, the material is slowly cooled. The slowest rate of cooling is achieved by leaving the material in the furnace and gradually reducing the temperature. This is an expensive process, since it ties up the furnace. More commonly, the material is cooled in air. Heat treatments of steel and aluminum are discussed in Chapters 3 and 4.

The grain size is affected by the rate of cooling. Rapid cooling limits the time available for grain growth, resulting in small grains, whereas slow cooling results in large grains.

2.2.4 ■ Alloys

The engineering characteristics of most metallic elements make them unsuitable for use in a pure form. In most cases, the properties of the materials can be significantly improved with the addition of alloying agents. An alloying

agent is simply a chemical that is compounded or in solution in the crystalline structure of a metal. Steel, composed primarily of iron and carbon, is perhaps the most common alloy. The way the alloying agent fits into the crystalline structure is extremely important. The alloying atoms can either fit into the voids between the atoms (*interstitial* atoms) or can replace the atoms in the lattice structure (*substitutional* atoms).

Because metals have a close pack structure, the radius of the interstitial atom must be less than 0.6 of the radius of the host element (Derucher et. al. 1994). Also, the solubility limit of interstitial atoms—less than 6%—is relatively low. Interstitial atoms can be larger than the size of the void in the lattice structure, but this will result in a strain of the structure and, therefore, will limit solubility.

If the characteristics of two metal elements are sufficiently similar, the metals can have complete miscibility; that is, there is no solubility limit. The atoms of the elements are completely interchangeable. The similarity criteria are defined by the Hume–Rothery rules (Shackelford 1996). Under these rules, the elements must have the following characteristics:

1. less than 15% difference in the atomic radius
2. the same crystal structure
3. similar electronegatives (the ability of the atom to attract an electron)
4. the same valence

Violation of any of the Hume–Rothery rules reduces the solubility of the atoms. Atoms that are either too large or too small will result in a strain of the lattice structure.

The arrangement of the alloy atoms in the structure can be either random or ordered. In a random arrangement, there is no pattern to the placement of the alloy atoms. An ordered arrangement can develop if the alloy element has a preference for a certain location in the lattice structure. For example, in a gold–copper alloy, an FCC structure, the copper preferably occupies the face positions and the gold preferably occupies the corner positions.

Frequently, more than one alloying agent is used to modify the characteristics of a metal. Steel is a good example of a multiple element alloy. By definition, steel contains iron with carbon; however, steel frequently includes other alloying elements such as chromium, copper, nickel, phosphorous, etc.

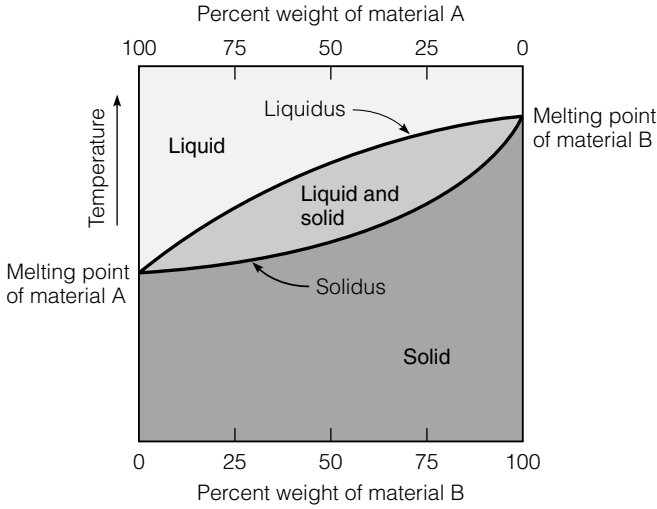
2.2.5 ■ Phase Diagrams

To produce alloys metals, the components are heated to a molten state, mixed, and then cooled. The temperature at which the material transitions between a liquid and a solid is a function of the percentages of the components. The liquid and solid states of a material are called *phases*, and a phase diagram displays the relationship between the percentages of the elements and the transition temperatures.

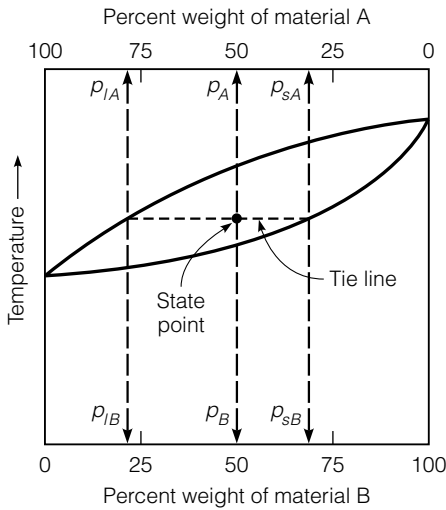
Soluble Materials The simplest type of phase diagram is for two elements that are completely soluble in both the liquid and solid phases. Solid solutions occur when the elements in the alloys remain dispersed throughout the matrix of the material in the solid state. A two-element or *binary* phase diagram is shown in Figure 2.14(a) for two completely soluble elements. In this diagram, temperature is plotted on the vertical axis and the percent weight of each element is plotted on the horizontal axis. In this case, the top axis is used for element A and the bottom axis is used for element B. The percentage of element B increases linearly across the axis, while the percentage of element A starts at 100% on the left and decreases to 0% on the right. Since this is a binary phase diagram, the sum of the percentage of elements A and B must equal 100%. In Figure 2.14(a), there are three areas. The areas at the top and bottom of the diagram have a single phase of liquid and solid material, respectively. Between the two single-phase areas, there is a two-phase area where the material is both liquid and solid. The line between the liquid and two-phase areas is the *liquidus*, and the line between the two-phase area and the solid area is the *solidus*. For a given composition of elements A and B, the liquidus defines the temperature at which, upon cooling, the first solid crystals form. The solidus defines the temperature at which all material has crystallized. It should be noted that, for a pure element, the transition between liquid and solid occurs at a single temperature. This is indicated on the phase diagram by the convergence of the liquidus and solidus on the left and right sides of Figure 2.14(a), where there is pure element A and B, respectively.

A specific composition of elements at a specific temperature is defined as the *state point*, as shown in Figure 2.14(b). If the state point is above the liquidus, all the material is liquid and composition of the liquid is the same as the total composition of the material. Similarly, if the state point is below the solidus, all the material is solid and the composition of the solid is the same as for the material. In the two-phase region between the liquidus and the solidus, the percent of material that is in either the liquid or solid phase varies with the temperature. In addition, the composition of the liquid and solid phases in this region changes with temperature. The compositions of the liquid and solid can be determined directly from the phase diagram by using the lever rule. First a *tie line* is established by connecting the liquidus and solidus with a horizontal line that passes through the state point, as shown on Figure 2.14(b). A vertical projection from the intersection of the tie line and the liquidus defines the composition of the liquid phase. A vertical projection from the intersection of the tie line and the solidus defines the composition of the solid phase. For the example in Figure 2.14(b), the alloy is composed of 50% material A and 50% material B. For the defined state point, 79% of the liquid material is element A and 21% is element B, and 31% of the solid phase material is element A and 69% of the solid material is element B. In addition, the percent of the material in the liquid and solid phases can be determined from the phase diagram. From mass balance, the total material must equal the sum of the masses of the components; that is,

$$m_t = m_l + m_s \quad (2.4)$$



(a)



(b)

FIGURE 2.14 Binary phase diagram, two soluble elements.

where

m_t = mass of total material

m_l = mass of the total material that is in the liquid phase

m_s = mass of the total material that is in the solid phase

This mass balance also applies to each of the component materials; that is,

$$p_B m_t = p_{lB} m_l + p_{sB} m_s \quad (2.5)$$

where

p_{lB} = percent of the liquid phase that is composed of material B

p_{sB} = percent of the solid phase that is composed of material B

p_B = percent of the material that is component B

From these two equations, the amount of material in the liquid and solid phase can be derived as

$$p_B m_t = p_{lB} m_l + p_{sB} (m_t - m_l)$$

$$p_B m_t = p_{lB} m_l + p_{sB} m_t - p_{sB} m_l$$

$$p_B m_t - p_{sB} m_t = p_{lB} m_l - p_{sB} m_l$$

$$m_l = \frac{(p_B - p_{sB})}{(p_{lB} - p_{sB})} m_t$$

$$m_s = m_t - m_l \quad (2.6)$$

Sample Problem 2.3

Considering an alloy of the two soluble components A and B described by a phase diagram similar to that shown in Figure 2.14, determine the masses of the alloy that are in the liquid and solid phases at a given temperature if the total mass of the alloy is 100 grams, component B represents 40% of the alloy, 20% of the liquid is component B, and 70% of solid is component B.

Solution

$$m_t = 100 \text{ g}$$

$$p_B = 40\%$$

$$p_{lB} = 20\%$$

$$p_{sB} = 70\%$$

From Equations 2.4 and 2.5,

$$m_l + m_s = 100$$

$$20m_l + 70m_s = 40 \times 100$$

Solving the two equations simultaneously, we get

$$m_l = \text{mass of the alloy that is in the liquid phase} = 60 \text{ g}$$

$$m_s = \text{mass of the alloy that is in the solid phase} = 40 \text{ g}$$

The same answer can also be obtained using Equation 2.6.

Insoluble Materials The discussion thus far has dealt with two completely soluble materials. It is equally important to understand the phase diagram for immiscible materials, that is, for components that are so dissimilar that

their solubility in each other is nearly negligible in the solid phase. Figure 2.15 shows the phase diagram for this situation. The intersections of the liquidus with the right and left axes are the melting points for each of the components. As the materials are blended together, the liquidus forms a V shape. The point of the V defines the combination of the components that will change from liquid to solid without the formation of two phases. This point defines the eutectic temperature and the eutectic composition for the components. The solidus is horizontal and passes through the eutectic temperature. There are two areas on the graph with two phases. The area to the left of the eutectic composition will have solid component A, and the liquid phase will be a mixture of the A and B components (vice versa for the area to the right of the eutectic composition).

The sudden phase transformation at the eutectic temperature means that the grains do not have time to grow as the material cools. Thus, the eutectic material will have a fine grain structure.

As the material cools at a temperature other than the eutectic temperature, one of the components becomes solid as the temperature drops below the liquidus. As a result, the amount of this component in the liquid continuously decreases as the material cools in the two-phase area. In fact, the composition of the liquid will follow the liquidus as the temperature decreases. When the eutectic temperature is reached, the remainder of the liquid becomes solid and has a fine grain structure. Starting with a composition of 80% B and 20% A, as in Figure 2.15, when the temperature is lowered to T , the liquid will contain 55% B and 45% A. The amount of each component in the liquid and solid phases can be determined by mass balance (Equations 2.4 and 2.5). At T , this will yield 44.4% liquid and 55.6% solid; all of the solid will be component B. Similarly, the composition of the solid at the eutectic temperature can be computed to be 66.7% solid B and 33.3% eutectic mixture. Since these materials are insoluble in the solid state, the

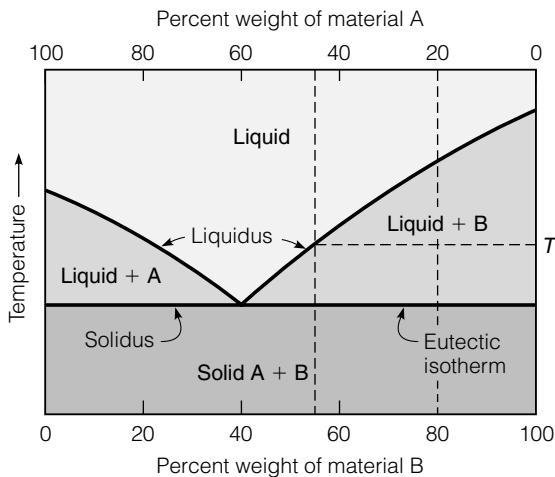


FIGURE 2.15 Binary phase diagram, insoluble solids.

eutectic mixture is composed of an intimate mixture of fine crystals of A and B, usually in a plate-like structure.

Partially Soluble Materials In between purely soluble and insoluble materials are the materials that are partially soluble. In other words, there is a solubility limit between the components of A and B. If the percent of component B is less than or equal to the solubility limit, on cooling all of the B atoms will be in solution with the A component. If the percent of the B component is above the solubility limit, the atoms in excess of the amount that will go into solution will form separate grains of the component B. The result is shown on the phase diagram in Figure 2.16. Note that the only difference between this phase diagram and the one shown in Figure 2.15 is the presence of the solid solution regions on each side of the graph. The composition analysis of the two-phase region is the same as that described for Figure 2.15.

Eutectoid Reaction Up to now, the phase diagram has been used to describe the transition between liquid and solid phases of materials. However, the lattice structure of some elements (e.g., iron) is a function of the temperature of the solid. As a result of the lattice transformation, the microstructure of the solid material changes as a function of temperature, as shown by the phase diagram in Figure 2.17. When this occurs, a eutectoid reaction occurs on the phase diagram; it has characteristics similar to the eutectic reaction, but is for a lattice structure transformation of the material rather than for a liquid–solid transformation. The rules for the analysis of the components of the material are the same as those discussed for the eutectic material. As with the phase transformation at the eutectic temperature, the transformation of the lattice structure at the eutectoid temperature will result in fine-grained materials.

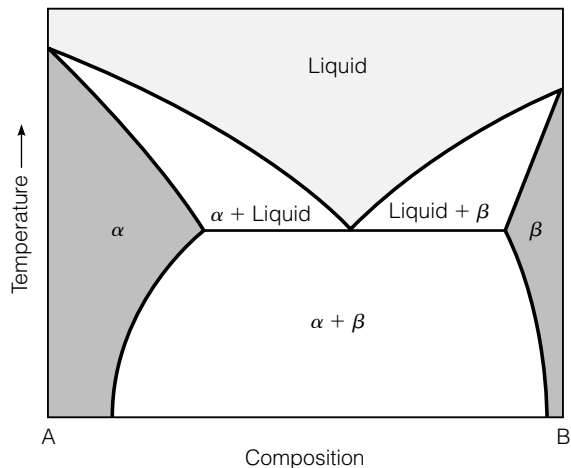


FIGURE 2.16 Binary phase diagram, partially soluble material.

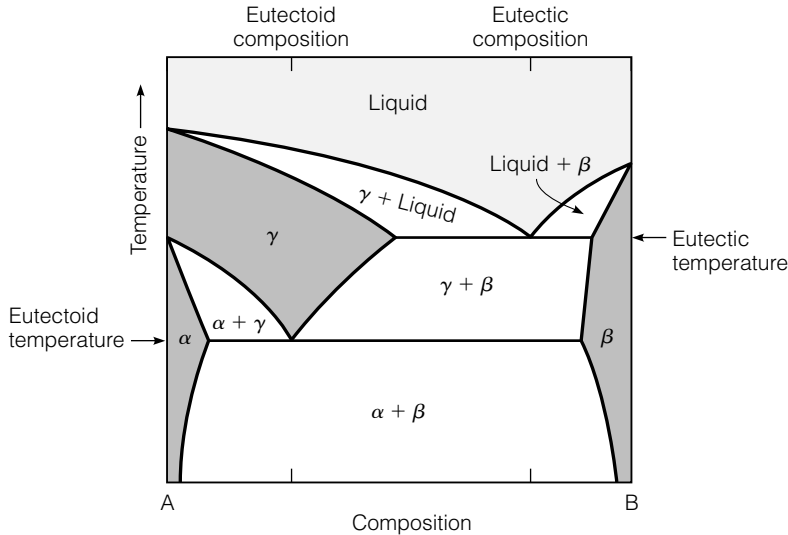


FIGURE 2.17 Phase diagram for a eutectoid reaction.

2.2.6 Combined Effects

The topics of lattice structure, grain size, heat treatments, and alloying are closely interrelated. The behavior of a metal is dictated by the combination of each of these factors. Clearly, the properties of a metal depend on the elemental make up, the refining and production process, and the types and extent of alloys used in the metal. These topics are too complex for detailed treatment in this text. A practicing engineer in this area must devote considerable study to material characteristics and the impact of alloying and heat treatments. Due to the importance of steel in civil engineering, the phase diagram and heat treatment of steel will be presented in Chapter 3.

2.3 Inorganic Solids

Inorganic solids include all materials composed of nonmetallic elements or a combination of metallic and nonmetallic elements. This class of materials is sometimes referred to as ceramic materials. By definition, ceramic elements have five, six, and seven valence electrons. Ceramic materials are formed by a combination of ionic and covalent bonds. In the generic sense, ceramics encompass a broad range of materials, including glass, pottery, inorganic cements, and various oxides. Fired clay products, including bricks and pottery, are some of the oldest ceramic products made by humans. In terms of tonnage, portland cement concrete is the most widely used manufactured material. In the 1980s, the search for highly durable products with

unique strength and thermal properties initiated the rapid development of advanced ceramics, such as zirconias, aluminum oxides, silicon carbides, and silicon nitrates. These materials have high strength, stiffness, and wear resistance, and are corrosion resistant. The high-performance or engineered ceramics have found many applications in machine and tool design. Although the availability of high-performance ceramics has grown rapidly, the civil and construction engineering applications of these materials have been generally limited due to the cost of the sophisticated ceramics and their lack of fracture toughness that is needed for structural design.

Five classes of ceramic materials have been defined (Ashby and Jones 1986):

1. glasses—based on silica
2. vitreous ceramics—clay products used for pottery, bricks, etc.
3. high-performance ceramics—highly refined inorganic solids used for specialty applications in which properties, not available from other materials, compensate for the high cost
4. cement and concrete—a multiphase material widely used in civil engineering applications
5. rocks and minerals

Ceramics can also be classified by the predominant type of their atomic bonding. Materials composed of a combination of nonmetallic and metallic elements have predominantly ionic bonds. Materials composed of two nonmetals have predominantly covalent bonds. The type of bond dictates the crystal structure of the compound. As with metals, inorganic solids have a well-defined, although more complicated, unit cell structure. This structure is repeated for the formation of the crystals.

In a simple ionic compound, the nonmetallic element will form either a face centered cubic or hexagonal close pack structure, as shown in Figure 2.18. Octahedral holes fit in the middle of the octahedron that is formed by connecting all the face atoms of the FCC unit cell. Tetrahedral holes are in the middle of the tetrahedron formed by connecting a corner atom with the adjacent face atoms. In the FCC structure, there is one octahedral hole and six tetrahedral holes. The number of atoms and the way

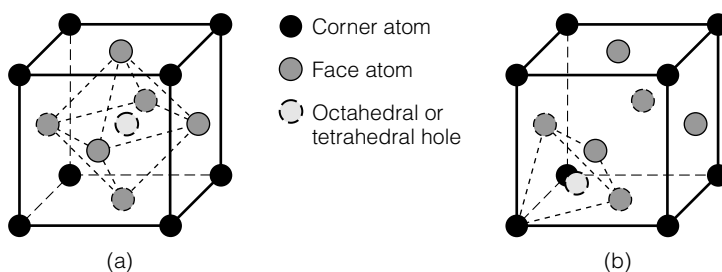


FIGURE 2.18 Lattice structure for simple ionic bonded ceramic materials: (a) octahedral hole and (b) tetrahedral hole.

they fit into the holes in the close pack structure is determined by the number of valence electrons in the metallic and nonmetallic elements. The atoms pack to maximize density, with the constraint that like ions are not nearest neighbors (Ashby and Jones 1986).

Simple covalent bonds form materials that are highly durable and strong. Diamond is the preeminent example of an elemental high-strength material. Diamond is widely used in industrial applications needing wear resistance, such as cutting tools. However, the most important of the predominately covalent bond materials used by civil engineers are the silicate compounds of portland cement concrete. Silicon atoms link with four oxygen atoms to form a stable tetrahedron (Figure 2.19) that is the basic building block for all silicates. When combined with metal oxides, MO, with a ratio of $MO:SiO_2$ of 2 or greater, the resulting silicate is made up of separate SiO_4 monomers linked by the MO molecules. Figure 2.20 shows the calcium silicate crystal structure. The primary reaction compounds of portland cement are tricalcium silicates and dicalcium silicates. Each of these have two or more metal oxides, CaO, per silicate molecule. An ionic bond forms between the metal oxide and the silicate.

Ceramics have only about one-fiftieth the fracture toughness of metals. Due to the nature of the ionic and covalent bonds, ceramic compounds tend to fracture in a brittle manner rather than to have plastic deformation, as is the case for metals. Like metals, ceramic compounds form distinct grains as a result of multiple nuclei forming multiple crystals during the production of the compound. In addition, due to the production process, ceramic materials tend to have internal cracks and flaws. Stress concentrations occur at the cracks, flaws, and grain boundaries, all of which lower the strength and toughness of the material. As a result, ceramic materials lack tensile strength and must be reinforced when they are used for structural applications.

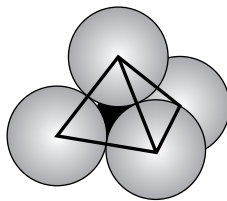


FIGURE 2.19 Silicate tetrahedron.

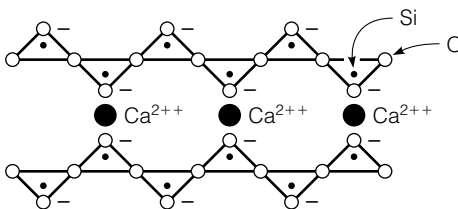


FIGURE 2.20 Simple inorganic solid structure (calcium silicate).

Glasses are a special type of inorganic solids; they do not develop a crystalline structure. Commercial glasses are based on silica. In glass, the silica tetrahedrons link at the corners, resulting in a random or amorphous structure. Glass can be made of pure silica; it has a high softening point and low thermal expansion. However, due to the high softening point, pure silica glass is hard to form. Metal oxides are added to reduce the cross-linking of the silicates, improving the ability to form the glass into the desired shape. Although glass has an amorphous structure, it is a very stable compound at atmospheric temperatures. It does not flow despite the often cited example of glass windows in European churches.

2.4 Organic Solids

All organic solids are composed of long molecules of covalent bonded carbon atoms. These molecules are chains of carbon and hydrogen combined with various radical components. The radical component can be a hydrogen atom, another hydrocarbon, or another element. These long molecules are bound together by secondary bonds; in many cases the molecules are also cross-linked with covalent bonds. There are a wide range of organic solids used in engineering. These can be classified as follows (Ashby and Jones 1986):

1. *Thermoplastics* are characterized by linear carbon chains that are not cross-linked; at low temperatures secondary bonds adhere the chains. Upon heating, the secondary bonds melt and the thermoplastics become a viscous material. Asphalt is a natural thermoplastic. It is obtained primarily by refining petroleum. In addition, there are many manufactured thermoplastics that have broad engineering applications. These include polyethylene, polypropylene, polytetrafluoroethylene, polystyrene, and polyvinyl chloride. Polyethylene and polypropylene are used in tubing, bottles, and electrical insulation. Polytetrafluoroethylene is commonly known as Teflon. In addition to cookware applications, Teflon is widely used for bearings and seals due to its very low friction and good adhesion characteristics. (Polytetrafluoroethylene is a carbon–fluorine chain.) Polystyrene is used for molded objects. It is foamed with carbon dioxide to make packing materials and thermal insulation. Polyvinyl chloride is used for low-pressure waterlines.
2. *Thermosets* are characteristically made of a resin and a hardener that chemically react to harden. In the formation of the solid, the carbon chains are cross-linked to form stable compounds that do not soften upon heating. The three generic types of thermosets are epoxy, polyester, and phenol–formaldehyde. Epoxies are used as glues and as the matrix material in plastic composites. Polyester is a fibrous material used in the reinforcing phase of fiberglass. Phenol–formaldehydes are brittle plastics such as bakelite and formica.

3. *Elastomers or rubbers* are characterized as linear polymers with limited cross-linking. At atmospheric temperatures the secondary bonds have melted. The cross-linking enables the material to return to its original shape when unloaded. Three forms of elastomers are polyisoprene (natural rubber), polybutadiene (synthetic rubber), and polychloroprene (Neoprene).
4. *Natural materials* are characterized as being grown in all plant matter. The primary material of interest is wood, which is composed of cellulose, lignin, and protein.

Other than the natural polymers, the balance of the organic solids are produced from refining and processing crude oil. In general, these products are classified as plastics. The properties of these materials are highly variable. Mechanical properties depend on the length of the polymer chains, the extent of cross-linking, and the type of radical compound. All of these factors can be controlled and altered in the production process to alter the material properties.

2.4.1 ■ Polymer Development, Structure, and Cross-Linking

The physical structure of the polymer chain grossly affects the mechanical response of plastics. The word polymer literally means multiple “mer” units. The mer is a base molecule that can be linked together to form the polymers. Figure 2.21 shows the structure of a simple ethylene molecule and the development of a polymer. The square boxes are carbon atoms and the open circles are hydrogen. Forming the polymer requires breaking the double bond, activating the monomer, and allowing it to link to others to form a long chain. The end of the chain either links to other chains or a terminator molecule, such as OH. Chains with useful mechanical properties require at least 500 monomers. The number of monomers in the chain defines the degree of polymerization; commercial polymers have a degree of polymerization of 10^3 to 10^5 .

The complexity of the linear chain polymer is increased by replacing hydrogen atoms with side groups or radicals, as shown in Figure 2.21. The radicals or side groups can be aligned on one side of the chain (*isotactic*), symmetrically on alternate sides of the chain (*syndiotactic*), or in a random fashion (*atactic*). The radicals can range from simple to complex molecules. For example, polyvinyl chloride has a Cl radical, polypropylene has a CH_3 radical, and polystyrene has C_6H_5 . The ability of the polymer chains to stack together is determined by the arrangement of the side chains. The simple chains can fold together into an orderly arrangement, whereas the complex side groups prevent stacking, leading to the amorphous nature of these materials.

More complex linear polymers are formed when two of the hydrogen atoms are replaced by different radicals. Polymethylmethacrylate, (Plexiglas), has the radicals CH_3 and COOCH_3 . As the complexity of the radicals and the

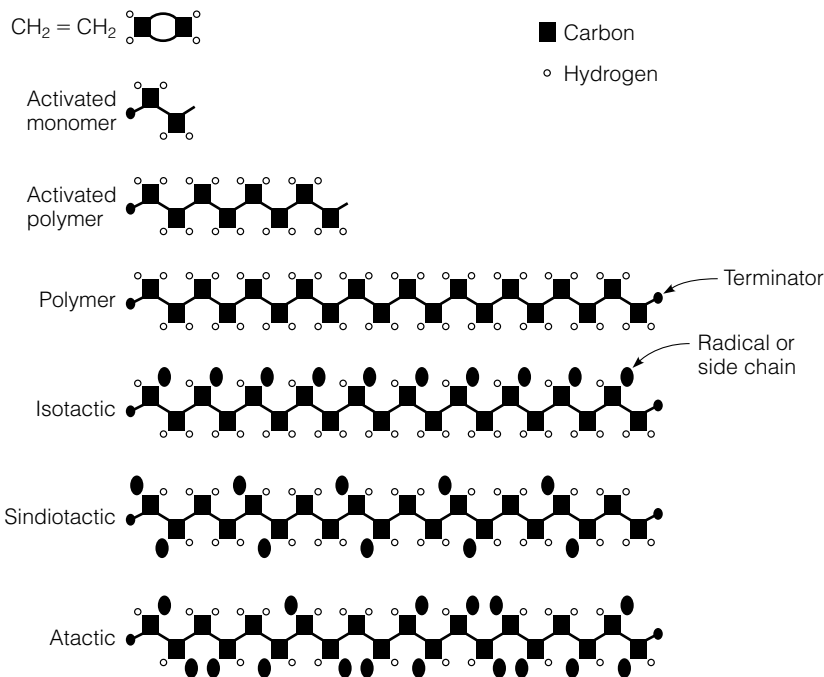


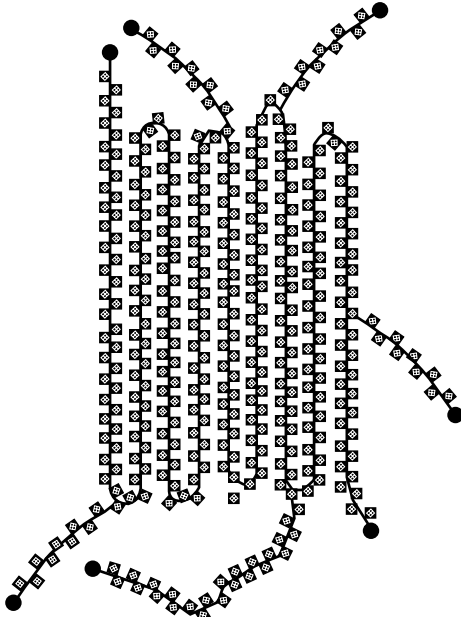
FIGURE 2.21 Polymer structures.

substitution for hydrogen increase, it becomes more difficult to form regular patterns.

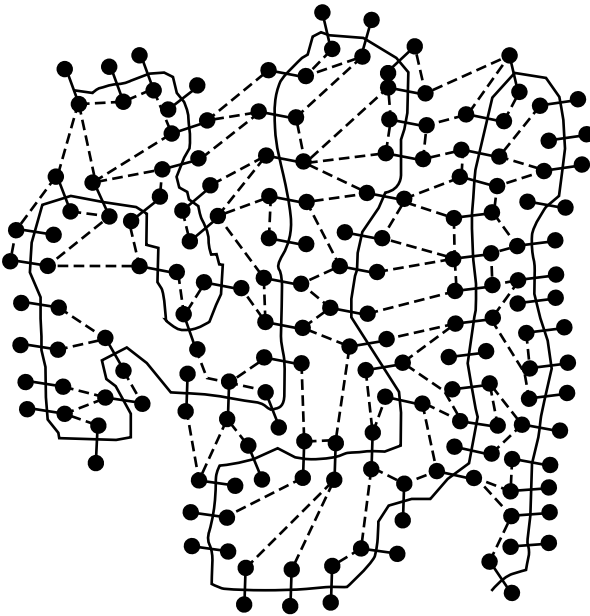
In thermoplastics, formed from linear polymers, the structure of the molecules is a blend of amorphous and crystal structure. When there are few side groups, an ordered structure is produced [Figure 2.22(a)]. As the number of side groups increases, the structure becomes increasingly random and cross-links develop. These structures are shown in Figure 2.22(b). Thermosets are formed from polyfunctional monomers. They are formed in a condensation reaction; in essence, the reaction bonds two chains together. Since the chains are formed from polyfunctional crystals, they have an amorphous structure with extensive cross-linking. Elastomers are formed with linear chains that have a limited number of cross-links.

2.4.2 Melting and Glass Transition Temperature

The reaction of polymers to temperature depends on the degree to which the material has crystallized. Highly ordered polymers have a fairly well-defined transition between elastic and viscous behavior. As the percent of crystallization decreases, the melting point is not well defined. However, the point at which these polymers transition to a glass phase is well defined. At elevated temperatures, the motion of the molecules forces a



(a)



(b)

FIGURE 2.22 Polymer chain structures: (a) ordered structure of linear polymer and (b) cross linked structure of linear polymer.

separation between them, resulting in a volume that is greater than required for tightly packed, motionless molecules. This excess volume is termed the *free volume*. As the material cools, the motion of the molecules is reduced and the viscosity increases. At a sufficiently low temperature, the molecules are no longer free to rearrange; thus, their position is fixed and the free volume becomes zero. This is the glass transition temperature. Below this temperature, the secondary bonds bind the material into an amorphous solid; above this temperature, the material behaves in an elastic manner. Figure 2.23 illustrates the concept of melting and glass transition temperatures, T_m and T_g . The glass transition temperature of Plexiglass is 100°C ; at room temperature it is a brittle solid. Above T_g , it becomes leathery and then rubbery. T_g for natural rubber is -70°C ; it is flexible at all atmospheric temperatures. However, when frozen, say in liquid nitrogen, it becomes a brittle solid.

2.4.3 Mechanical Properties

The mechanical behavior of polymers is directly related to the degree of orientation of the molecules and the amount of cross-linking by covalent bonds. The modulus of a polymer is the average of the stiffness of the bonds.

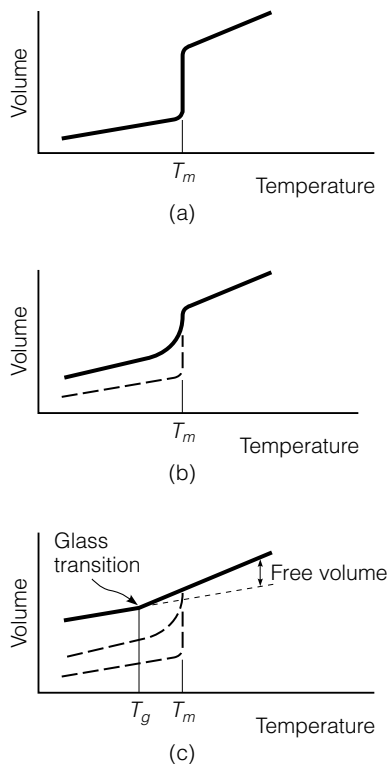


FIGURE 2.23 Melting point and glass transition temperatures: (a) perfect crystallization, (b) imperfect crystallization, and (c) glass formation.

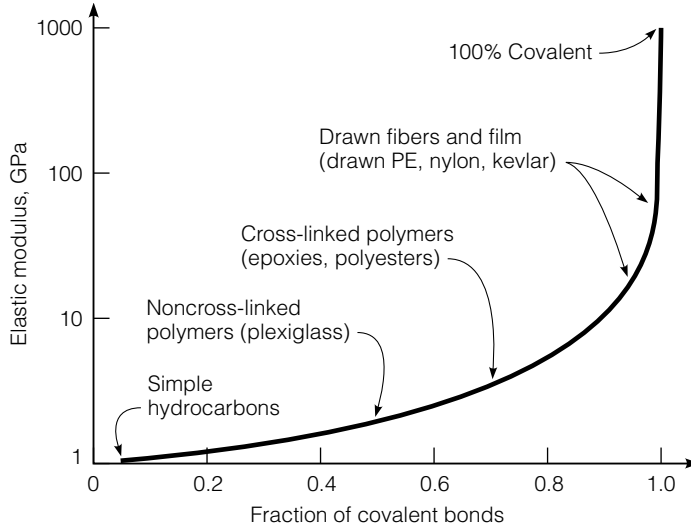


FIGURE 2.24 Modulus of polymers.

The modulus can be estimated from volume fractions of the covalent bonding as (Ashby and Jones 1986)

$$\varepsilon = f \frac{\sigma}{E_1} + (1 - f) \frac{\sigma}{E_2} \tag{2.7}$$

where

- ε = strain of the material
- σ = stress of the material
- f = fraction of covalent bonds
- E_1 = stiffness of covalent bonds, about 1000 GPa for diamond
- E_2 = stiffness of secondary bonds, about 1 GPa for paraffin wax

Based on Equation 2.7, the expected modulus of various polymers can be computed for temperatures less than the glass transition temperature, as shown in Figure 2.24.

S U M M A R Y

The behavior of materials important to engineering is directly related to their microscopic and macroscopic structure. Although our understanding of these materials is imperfect at this time, much of their behavior can be attributed

to the bonding and arrangement of the materials at the atomic level. This chapter provides only a broad overview of the subject. For more information, consult references with more in-depth treatments of these subjects.

Q U E S T I O N S A N D P R O B L E M S

- 2.1 Define elastic and plastic behaviors at the micro and macro levels.
- 2.2 Describe the parts of an atom. Define proton, electron, atomic number, and atomic mass.
- 2.3 What are the valence electrons and why are they important?
- 2.4 Describe the order in which electrons fill the shells and subshells.
- 2.5 Describe the different types of bonds.
- 2.6 Why do atoms maintain specific separations?
- 2.7 Materials are generally classified into three categories based on the predominant types of bond. What are these three categories and what are the predominant types of bond in each category? For each category, provide two examples of common materials used by civil engineers.
- 2.8 What is the atomic packing factor? What information do you need to compute it?
- 2.9 Describe FCC, BCC, and HCP lattice structures.
- 2.10 Show for the face-center cubic crystal structure that the unit cell edge length a and the atomic radius r are related through $a = 2\sqrt{2}r$.
- 2.11 Referring to the BCC lattice structure shown in Figure P2.11

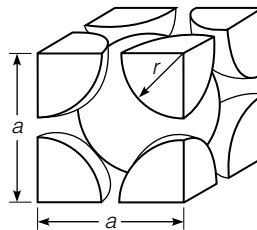


FIGURE P2.11

- a. Determine the number of equivalent whole atoms in the unit cell.
- b. Calculate the relation between a and r .
- c. Calculate the atomic packing factor of the BCC lattice structure if the volume of the sphere is $4/3\pi r^3$.

- 2.12 Calculate the volume of the unit cell of iron in cubic meters, given that iron has a body-center cubic crystal structure and an atomic radius of 0.124 nm.
- 2.13 If aluminum has an FCC crystal structure and an atomic radius of 0.143 nm, calculate the volume of its unit cell in cubic meters.
- 2.14 Using the information available in Table 2.3, calculate the volume of the unit cell of copper in cubic meters.
- 2.15 Calculate the density of iron, given that it has a BCC crystal structure, an atomic radius of 0.124 nm, and an atomic mass of 55.9 g/mole.
- 2.16 Calculate the density of aluminum, given that it has an FCC crystal structure, an atomic radius of 0.143 nm, and an atomic mass of 26.98 g/mole.
- 2.17 Calculate the radius of the copper atom, given that copper has an FCC crystal structure, a density of 8.89 g/cm³, and an atomic mass of 63.55 g/mole.
- 2.18 What are the classes of defects in crystal structures?
- 2.19 Why do grains form in crystal structures?
- 2.20 Explain the slipping of atoms and the effect on material deformation.
- 2.21 Sketch a phase diagram for two soluble components.
- 2.22 What is the eutectic composition and why is it important?
- 2.23 Considering an alloy of the two soluble components A and B described by a phase diagram similar to that shown in Figure 2.14, determine the masses of the alloy that are in the liquid and solid phases at a given temperature if the total mass of the alloy is 100 grams, component B represents 65% of the alloy, 30% of the liquid is component B, and 80% of solid is component B.
- 2.24 Considering an alloy of the two soluble components A and B described by a phase diagram similar to that shown in Figure 2.14, determine the masses of the alloy that are in the liquid and solid phases at a given temperature if the total mass of the alloy is 100 grams, component B represents 45% of the alloy, 17% of the liquid is component B, and 65% of solid is component B.
- 2.25 Figure P2.25 shows a portion of the H₂O–NaCl phase diagram.
 - a. Using the diagram, briefly explain how spreading salt on ice causes the ice to melt. Show numerical examples in your discussion.
 - b. At a salt composition of 10%, what is the temperature at which ice will start melting?
 - c. What is the eutectic temperature of the ice and salt combination?
- 2.26 What are the five classes of ceramic materials?
- 2.27 What are the four types of organic solids used in engineering applications? Define each one and give examples.

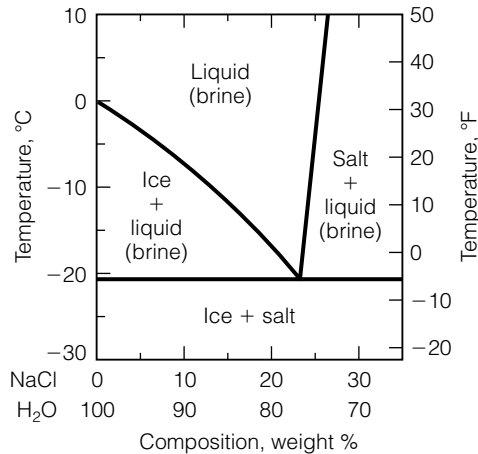


FIGURE P2.25

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3

STEEL



The use of iron dates back to about 1500 B.C. when primitive furnaces were used to heat the ore in a charcoal fire. Ferrous metals were produced on a relatively small scale until the blast furnace was developed in the 18th century. Iron products were widely used in the latter half of the 18th century and the early part of the 19th century. Steel production started in mid-1800s when the Bessemer converter was invented. In the second half of the 19th century, steel technology advanced rapidly due to the development of the basic oxygen furnace and continuous casting methods. More recently, computer-controlled manufacturing has increased the efficiency and reduced the cost of steel production.

Currently, steel and steel alloys are used widely in civil engineering applications. In addition, wrought iron is still used on a smaller scale for pipes, as well as for general blacksmith work. Cast iron is used for pipes, hardware, and machine parts not subjected to tensile or dynamic loading.

Steel products used in construction can be classified as follows:

1. *structural steel* for use in plates, bars, pipes, structural shapes, etc. (Figure 3.1)
2. *fastening products* used for structural connections, including bolts, nuts and washers
3. *reinforcing steel* (rebars) for use in concrete reinforcement (Figure 3.2)
4. miscellaneous products for use in such applications as forms and pans

Civil and construction engineers rarely have the opportunity to formulate steel with specific properties. Rather, they must select existing products from suppliers. Even the shapes for structural elements are generally restricted to those readily available from manufacturers. While specific shapes can be made to order, the cost to fabricate low-volume members is generally prohibitive. Therefore, the majority of civil engineering projects are designed using standard steel types and structural shapes.

Even though civil and construction engineers are not responsible for formulating steel products, they still must understand how steel is manufactured



FIGURE 3.1 Truss made of structural steel for the structural support of a building.

and treated and how it responds to loads and environmental conditions. This chapter reviews steel production, the iron–carbon phase diagram, heat treatment, steel alloys, structural steel, steel fasteners, and reinforcing steel. The chapter also presents common tests used to characterize the mechanical properties of steel. The topics of welding and corrosion of steel are also introduced.

3.1 Steel Production

The overall process of steel production is shown in Figure 3.3. This process consists of the following three phases:

1. reducing iron ore to pig iron
2. refining pig iron to steel
3. forming the steel into products

The materials used to produce pig iron are coal, limestone, and iron ore. The coal, after transformation to coke, supplies carbon used to reduce iron



FIGURE 3.2 Steel rebar used to reinforce portland cement concrete wall.

oxides in the ore. Limestone is used to help remove impurities. Prior to reduction, the concentration of iron in the ore is increased by crushing and soaking the ore. The iron is magnetically extracted from the waste, and the extracted material is formed into pellets and fired. The processed ore contains about 65% iron.

Reduction of the ore to pig iron is accomplished in a blast furnace. The ore is heated in the presence of carbon. Oxygen in the ore reacts with carbon to form gases. A flux is used to help remove impurities. The molten iron, with an excess of carbon in solution, collects at the bottom of the furnace. The impurities, slag, float on top of the molten pig iron.

The excess carbon, along with other impurities, must be removed to produce high-quality steel. Using the same refining process, scrap steel can be recycled. Three types of furnaces are used for refining pig iron to steel:

1. open hearth
2. basic oxygen
3. electric arc

The open hearth and basic oxygen furnaces remove excess carbon by reacting the carbon with oxygen to form gases. Lances circulate oxygen through the molten material. The process is continued until all impurities are removed and the desired carbon content is achieved. Open hearth furnaces have been used since the early 1900s. Now, due to greater efficiency and productivity,

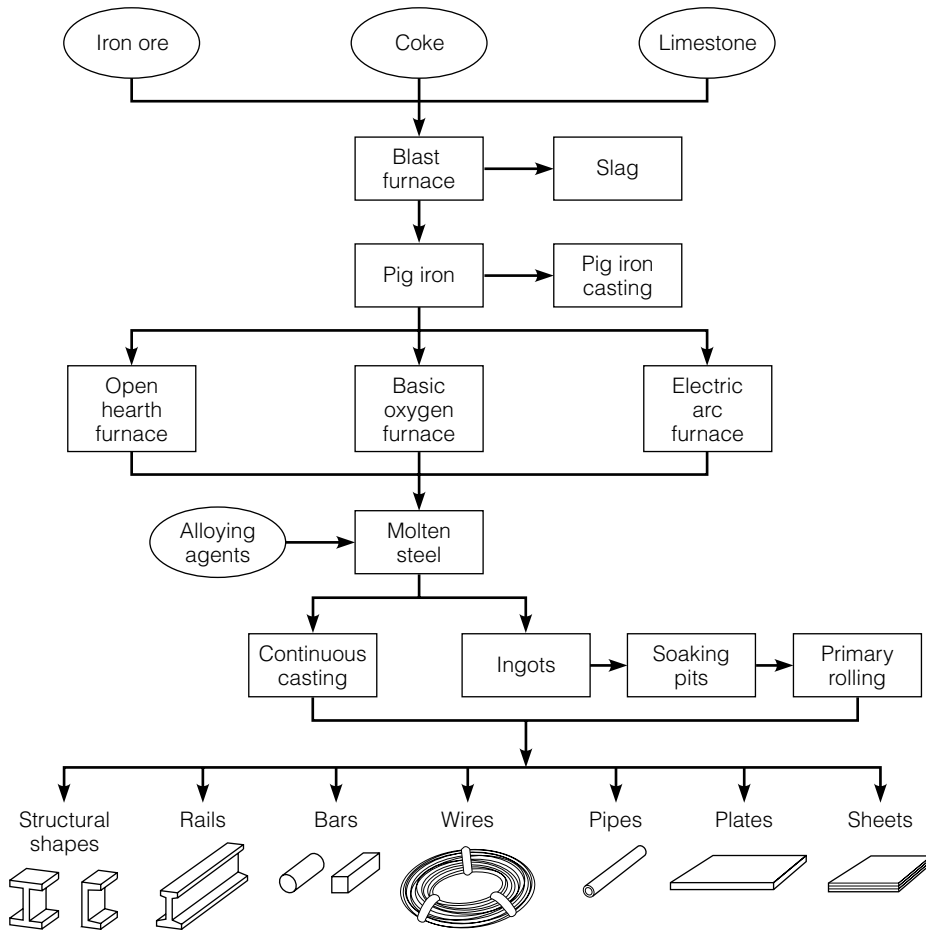


FIGURE 3.3 Conversion of raw material into different steel shapes.

basic oxygen furnaces are the industry standard for high-production mills. A basic oxygen furnace can refine 280,000 kg (300 tons) of steel in 25 minutes, compared with the eight hours it takes to refine the same quantity of steel in an open hearth furnace.

Electric furnaces use an electric arc between carbon electrodes to melt and refine the steel. These plants require a tremendous amount of energy and are primarily used to recycle scrap steel. Electric furnaces are frequently used in minimills, which produce a limited range of products. In this process, molten steel is transferred to the ladle. Alloying elements and additional agents can be added either in the furnace or the ladle.

During the steel production process, oxygen may become dissolved in the liquid metal. As the steel solidifies, the oxygen can combine with carbon to form carbon monoxide bubbles that are trapped in the steel and can act as initiation points for failure. Deoxidizing agents, such as aluminum, ferrosilicon

and manganese, can eliminate the formation of the carbon monoxide bubbles. Completely deoxidized steels are known as *killed steels*. Steels that are generally killed include

- Those with a carbon content greater than 0.25%
- All forging grades of steels
- Structural steels with carbon content between 0.15 and 0.25 percent
- Some special steel in the lower carbon ranges

Regardless of the refining process, the molten steel, with the desired chemical composition, is then either cast into ingots (large blocks of steel) or cast continuously into a desired shape. Continuous casting is becoming the standard production method, since it is more energy efficient than casting ingots, as the ingots must be reheated prior to shaping the steel into the final product.

3.2 Iron–Carbon Phase Diagram

In refining steel from iron ore, the quantity of carbon used must be carefully controlled in order for the steel to have the desired properties. The reason for the strong relationship between steel properties and carbon content can be understood by examining the iron–carbon phase diagram.

Figure 3.4 presents a commonly accepted iron–carbon phase diagram. One of the unique features of this diagram is that the abscissa extends only to 6.7% rather than 100%. This is a matter of convention. In an iron-rich material, each carbon atom bonds with three iron atoms to form iron carbide, Fe_3C , also called cementite. Iron carbide is 6.7% carbon by weight. Thus, on the phase diagram, a carbon weight of 6.7% corresponds to 100% iron carbide. A complete iron–carbon phase diagram should extend to 100% carbon. However, only the iron rich portion, as shown on Figure 3.4, is of practical significance (Callister 2003). In fact, structural steels have a maximum carbon content of less than 0.3%, so only a very small portion of the phase diagram is significant for civil engineers.

The left side of Figure 3.4 demonstrates that pure iron goes through two transformations as temperature increases. Pure iron below 912°C has a BCC crystalline structure called ferrite. At 912°C the ferrite undergoes a polymorphic change to a FCC structure called austenite. At 1394°C, another polymorphic change occurs, returning the iron to a BCC structure. At 1539°C the iron melts into a liquid. The high and low temperature ferrites are identified as δ and α ferrite, respectively. Since δ ferrite occurs only at very high temperatures, it does not have practical significance for this book.

Carbon goes into solution with α ferrite at temperatures between 400°C and 912°C. However, the solubility limit is very low, with a maximum of 0.022% at 727°C. At temperatures below 727°C and to the right of the solubility limit line, α ferrite and iron carbide coexist as two phases. From 727°C to 1148°C, the solubility of carbon in the austenite increases from 0.77% to

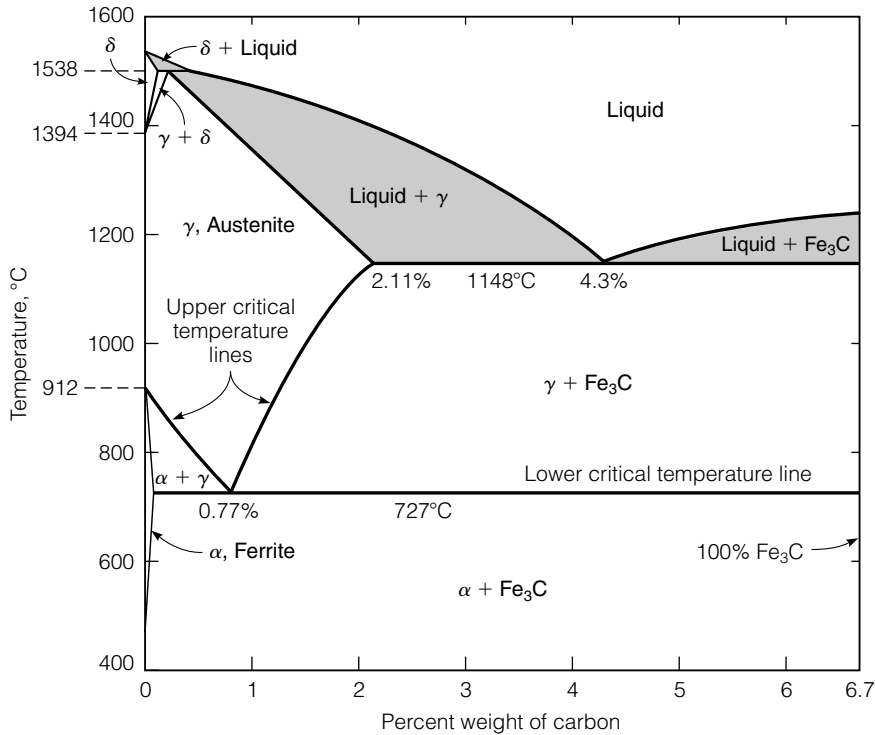


FIGURE 3.4 The iron–iron carbide phase diagram.

2.11%. The solubility of carbon in austenite is greater than in a ferrite because of the crystalline structure of the austenite.

At 0.77% carbon and 727°C, a eutectoid reaction occurs; that is, a solid phase change occurs when either the temperature or carbon content changes. At 0.77% carbon, and above 727°C, the carbon is in solution as an interstitial element, within the FCC structure of the austenite. A temperature drop to below 727°C, which happens slowly enough to allow the atoms to reach an equilibrium condition, results in a two-phase material, a ferrite and iron carbide. The α ferrite will have 0.022% carbon in solution, and the iron carbide will have a carbon content of 6.7%. The ferrite and iron carbide will form as thin plates, a lamellae structure. This eutectoid material is called pearlite.

At carbon contents less than the eutectoid composition, 0.77% carbon, *hypoeutectoid* alloys are formed. Consider a carbon content of 0.25%. Above approximately 860°C, solid austenite exists with carbon in solution. The austenite consists of grains of uniform material that were formed when the steel was cooled from a liquid to a solid. Under equilibrium temperature drop from 860°C to 727°C, α ferrite is formed and accumulates at the grain boundaries of the austenite. This is a proeutectoid ferrite. At temperatures slightly above 727°C, the ferrite will have 0.022% carbon in solution and austenite will have 0.77% carbon. When the temperature drops below 727°C,

the austenite will transform to pearlite. The resulting structure consists of grains of pearlite surrounded by a skeleton of α ferrite.

When the carbon content is greater than the eutectoid composition, 0.77% carbon, hypereutectoid alloys are formed. Iron carbide forms at the grain boundaries of the austenite at temperatures above 727°C. The resulting microstructure consists of grains of pearlite surrounded by a skeleton of iron carbide.

The lever rule for the analysis of phase diagrams can be used to determine the phases and constituents of steel.

Sample Problem 3.1

Calculate the amounts and compositions of phases and constituents of steel composed of iron and 0.25% carbon just above and below the eutectoid isotherm.

Solution

At a temperature just higher than 727°C, all the austenite will have a carbon content of 0.77% and will transform to pearlite. The ferrite will remain as primary ferrite. The proportions can be determined by using the lever rule:

Primary α : 0.022% C,

$$\text{Percent primary } \alpha = \left[\frac{0.77 - 0.25}{0.77 - 0.022} \right] \times 100 = 69.5\%$$

$$\text{Percent pearlite} = \left[\frac{0.25 - 0.022}{0.77 - 0.022} \right] = 30.5\%$$

At a temperature just below 727°C, the phases are ferrite and iron carbide. The ferrite will have 0.022% carbon, so we have

$$\text{Percent ferrite, } \alpha: (0.022\% \text{ C}) = \left[\frac{6.67 - 0.25}{6.67 - 0.022} \right] \times 100 = 96.6\%$$

$$\text{Percent pearlite} = \left[\frac{0.25 - 0.022}{6.67 - 0.022} \right] = 3.4\%$$

Figure 3.5 shows an optical photomicrograph of a hot-rolled mild steel plate with a carbon content of 0.18% by weight that was etched with 3% nitol. The photomicrograph is magnified at 50 \times . The light etching phase is proeutectoid ferrite and the dark constituent is pearlite. Note the banded structure resulting from the rolling processes. Figure 3.6 shows the same material as Figure 3.5, except that the magnification is 400 \times . At this magnification, the alternating layers of ferrite and cementite in the pearlite can be seen.

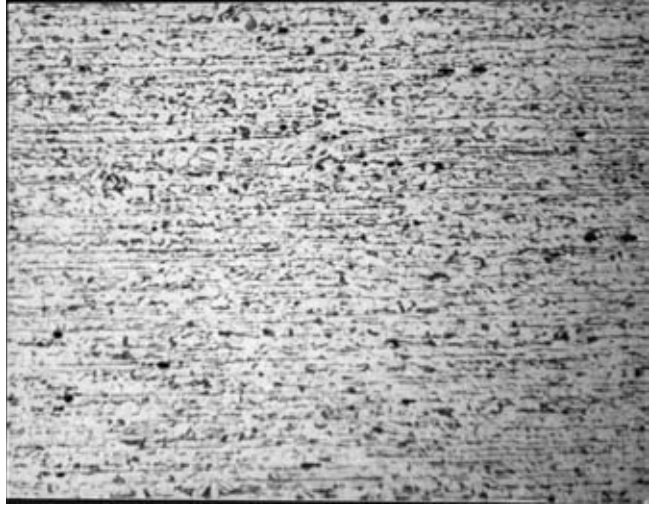


FIGURE 3.5 Optical photomicrograph of hot rolled mild steel plate (magnification: 50x).

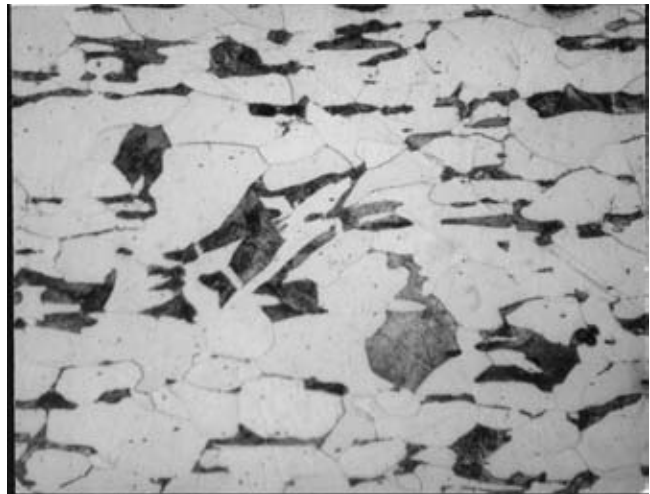


FIGURE 3.6 Optical photomicrograph of hot rolled mild steel plate (magnification: 400x).

The significance of ferrite, pearlite, and iron carbide formation is that the properties of the steel are highly dependent on the relative proportions of ferrite and iron carbide. Ferrite has relatively low strength but is very ductile. Iron carbide has high strength but has virtually no ductility. Combining these materials in different proportions alters the mechanical properties of the steel. Increasing the carbon content increases strength and hardness, but reduces ductility. However, the modulus of elasticity of steel does not change by altering the carbon content.

All of the preceding reactions are for temperature reduction rates that allow the material to reach equilibrium. Cooling at more rapid rates greatly

alters the microstructure. Moderate cooling rates produce bainite, a fine-structure pearlite without a proeutoid phase. Rapid quenching produces martensite; the carbon is supersaturated in the iron, causing a body center tetragonal lattice structure. Time–temperature transformation diagrams are used to predict the structure and properties of steel subjected to heat treatment. Rather than going into the specifics, the different types of heat treatments are described.

3.3 Heat Treatment of Steel

Properties of steel can be altered by applying a variety of heat treatments. For example, steel can be hardened or softened by using heat treatment; the response of steel to heat treatment depends upon its alloy composition. Common heat treatments employed for steel include annealing, normalizing, hardening, and tempering. The basic process is to heat the steel to a specific temperature, hold the temperature for a specified period of time, then cool the material at a specified rate. The temperatures used for each of the treatment types are shown in Figure 3.7.

3.3.1 Annealing

The objectives of annealing are to refine the grain, soften the steel, remove internal stresses, remove gases, increase ductility and toughness, and change

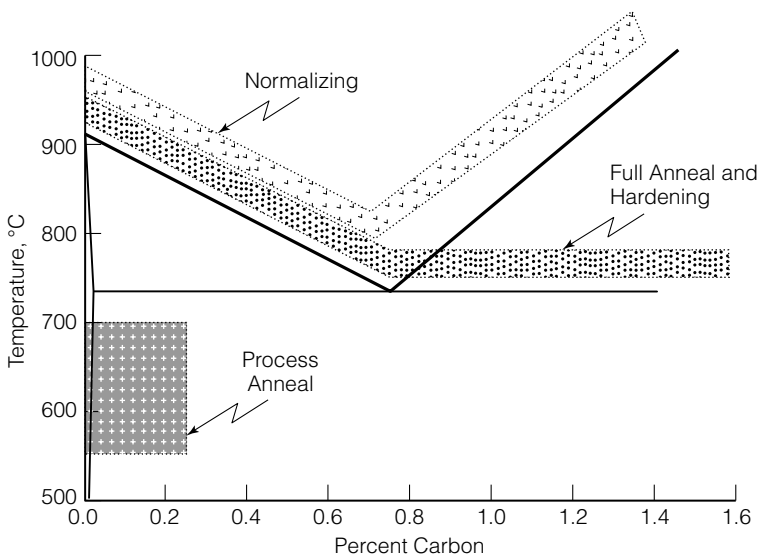


FIGURE 3.7 Heat treatment temperatures.

electrical and magnetic properties. Four types of annealing can be performed, depending on the desired results of the heat treatment:

Full annealing requires heating the steel to about 50°C above the austenitic temperature line and holding the temperature until all the steel transforms into either austenite or austenite–cementite, depending on the carbon content. The steel is then cooled at a rate of about 20°C per hour in a furnace to a temperature of about 680°C, followed by natural convection cooling to room temperature. Due to the slow cooling rate, the grain structure is a coarse pearlite with ferrite or cementite, depending on the carbon content. The slow cooling rate ensures uniform properties of the treated steel. The steel is soft and ductile.

Process annealing is used to treat work-hardened parts made with low carbon steel (i.e., less than 0.25 percent carbon). The material is heated to about 700°C and held long enough to allow recrystallization of the ferrite phase. By keeping the temperature below 727°C, there is not a phase shift between ferrite and austenite, as occurs during full annealing. Hence, the only change that occurs is refinement of the size, shape, and distribution of the grain structure.

Stress relief annealing is used to reduce residual stresses in cast, welded, and cold-worked parts and cold-formed parts. The material is heated to 600 to 650°C, held at temperature for about one hour, and then slowly cooled in still air.

Spheroidization is an annealing process used to improve the ability of high carbon (i.e., more than 0.6 percent carbon) steel to be machined or cold worked. It also improves abrasion resistance. The cementite is formed into globules (spheroids) dispersed throughout the ferrite matrix.

3.3.2 ■ Normalizing

Normalizing is similar to annealing, with a slight difference in the temperature and the rate of cooling. Steel is normalized by heating to about 60°C (110°F) above the austenite line and then cooling under natural convection. The material is then air cooled. Normalizing produces a uniform, fine-grained microstructure. However, since the rate of cooling is faster than that used for full annealing, the rate of cooling of shapes with varying thicknesses results in the normalized parts having less uniformity than could be achieved with annealing. Since structural plate has a uniform thickness, normalizing is an effective process and results in high fracture toughness of the material.

3.3.3 ■ Hardening

Steel is hardened by heating it to a temperature above the transformation range and holding it until austenite is formed. The steel is then quenched (cooled rapidly) by plunging it into, or spraying it with, water, brine, or oil. The rapid cooling “locks” the iron into a BCC structure, *martensite*, rather than allowing the transformation to the ferrite FCC structure. Martensite has a very hard and brittle structure. Since the cooling occurs more rapidly at

the surface of the material being hardened, the surface of the material is harder and more brittle than the interior of the element, creating nonhomogenous characteristics. Due to the rapid cooling, hardening puts the steel in a state of strain. This strain sometimes causes steel pieces with sharp angles or grooves to crack immediately after hardening. Thus, hardening must be followed by tempering.

3.3.4 ■ Tempering

The predominance of martensite in quench-hardened steel results in an undesirable brittleness. Tempering is performed to improve ductility and toughness. Martensite is a somewhat instable structure. Heating causes carbon atoms to diffuse from martensite to produce a carbide precipitate and formation of ferrite and cementite. After quenching, the steel is cooled to about 40°C then reheated by immersion in either oil or nitrate salts. The steel is maintained at the elevated temperature for about two hours and then cooled in still air.

3.3.5 ■ Example of Heat Treatment

In the quest to economically produce high-strength low-alloy steels, the industry has developed specifications for several new steel products, such as A913. This steel is available with yield stresses ranging from 50,000 to 75,000 psi. The superior properties of A913 steel are obtained by a quench-self-tempering process. Following the last hot rolling pass for shaping, for which the temperature is typically 850°C (1600°F), an intense water-cooling spray is applied to the surface of the beam to quench (rapidly cool) the skin. Cooling is interrupted before the core of the material is affected. The outer layers are then tempered as the internal heat of the beam flows to the surface. After the short cooling phase, the self-tempering temperature is 600°C (1100°F) (Bouchard and Axmann, 2000).

3.4 Steel Alloys

Alloy metals can be used to alter the characteristics of steel. By some counts, there are as many as 250,000 different alloys of steel produced. Of these, as many as 200 may be used for civil engineering applications. Rather than go into the specific characteristics of selected alloys, the general effect of different alloying agents will be presented. Alloy agents are added to improve one or more of the following properties:

1. hardenability
2. corrosion resistance
3. machineability
4. ductility
5. strength

TABLE 3.1 Common Steel Alloying Agents (Budinski, 1996) (Reprinted with permission of Prentice-Hall, Inc.)

	Typical Ranges in Alloy Steels (%)	Principal Effects
Aluminum	<2	Aids nitriding Restricts grain growth Removes oxygen in steel melting
Sulfur	<0.5	Adds machinability Reduces weldability and ductility
Chromium	0.3 to 0.4	Increases resistance to corrosion and oxidation Increases hardenability Increases high-temperature strength Can combine with carbon to form hard, wear-resistant microconstituents
Nickel	0.3 to 5	Promotes an austenitic structure Increases hardenability Increases toughness
Copper	0.2 to 0.5	Promotes tenacious oxide film to aid atmospheric corrosion resistance
Manganese	0.3 to 2	Increases hardenability Promotes an austenitic structure Combines with sulfur to reduce its adverse effects
Silicon	0.2 to 2.5	Removes oxygen in steel making Improves toughness Increases hardenability
Molybdenum	0.1 to 0.5	Promotes grain refinement Increases hardenability Improves high-temperature strength
Vanadium	0.1 to 0.3	Promotes grain refinement Increases hardenability Will combine with carbon to form wear-resistant microconstituents

Common alloy agents, their typical percentage range, and their effects are summarized in Table 3.1.

By altering the carbon and alloy content and by using different heat treatments, steel can be produced with a wide variety of characteristics.

These are classified as follows:

1. Low alloy
 - Low carbon
 - Plain
 - High strength–low alloy
 - Medium carbon
 - Plain
 - Heat treatable
 - High carbon
 - Plain
 - Tool
2. High Alloy
 - Tool
 - Stainless

Steels used for construction projects are predominantly low- and medium-carbon plain steels. Stainless steel has been used in some highly corrosive applications, such as dowel bars in concrete pavements and steel components in swimming pools and drainage lines. The Specialty Steel Industry of North America, SSINA, promotes the use of stainless steel for structural members where corrosion resistance is an important design consideration (SSINA, 1999).

The use and control of alloying agents is one of the most significant factors in the development of steels with better performance characteristics. The earliest specification for steel used in building and bridge construction, published in 1900, did not contain any chemical requirements. In 1991 ASTM published the specification which controls content of 10 alloying elements in addition to carbon (Hassett, 2003).

3.5 Structural Steel

Structural steel is used in hot-rolled structural shapes, plates, and bars. Structural steel is used for various types of structural members, such as columns, beams, bracings, frames, trusses, bridge girders, and other structural applications (see Figure 3.8).

3.5.1 Structural Steel Grades

Due to the widespread use of steel in many applications, there are a wide variety of systems for identifying or designating steel, based on grade, type and class. Virtually every country with an industrial capacity has specifications for steel. In the United States, there are several associations that write specifications for steel, such as the Society of Automotive Engineers, SAE, the American Iron and Steel Institute, AISI, and the American Society for Testing and Materials, ASTM. The most widely used designation system was developed cooperatively by SAE and AISI based on chemical composition (Key-to-Steel, 2005). However, the materials and products used in building design and construction in the United States are almost exclusively designated by ASTM specifications (Carter, 2004). ASTM specification names consist of a letter,



FIGURE 3.8 Structural steel used to make columns, beams, and floors for the structural support of a building.

generally an A for ferrous materials, followed by an arbitrary serially assigned number. For example, ASTM A7 was a specification for structural steel written in 1900 and ASTM A992 was published in 1999 (Carter, 2004). The designation or specification number does not contain any meaningful information other than to serve as a reference. Within ASTM specifications, the terms *grade*, *type*, and *class* are used in an inconsistent manner. In some ASTM steel specifications, the term grade identifies the yield strength, while in other specifications, the term grade can indicate requirements for both chemical compositions and mechanical properties. ASTM and SAE have developed the Unified Numbering System, UNS (ASTM E527), based on chemical composition. This system uses a letter to identify the broad class of alloys, and a five-digit number to define specific alloys within the class.

Several grades of structural steel are produced in the United States. Table 3.2 is a summary of selected information from various sources. The American Institute of Steel Construction, AISC, Manual for Steel Construction is an excellent reference on the types of steel used for structural applications. However, the best sources of information for structural steels are the various ASTM specifications. Of particular note is the fact that additional requirements are frequently included, dependent on the geometry of the product made with a particular steel.

Historically, dating back to 1900, only two types of structural steel were used in the United States: A7 for bridges and A9 for buildings. The specifications for these materials were very similar and in 1938, they were combined

TABLE 3.2 Designations, Properties, and Composition of ASTM Structural Steel

Steel Type	ASTM designation	F _y (psi)	F _u (psi)	Elon- gation ²	Typical Chemical Composition ¹														
					C	Cu ⁴	Mn	P	S	Ni	Cr	Si	Mo	V					
Carbon	A36	36	58–80	23	0.26	0.2	0.75	0.04	0.05										
	A500	A53 Gr. B	35	60		0.25	0.4	0.95	0.05	0.045	0.4	0.4			0.15	0.08			
		Gr. B	42	58	23	0.3	0.18		0.05	0.63									
	A501	Gr. C	46	62	21	0.27	0.18	1.4	0.05	0.063									
			50	62															
	High-strength Low-alloy	A529	36	58	23	0.3	0.18		0.15	0.63									
		Gr.50	50	65–100	19	0.27	0.2	1.35	0.04	0.05									
			Gr.55	55	70–100														
		A572	Gr. 42	42	60	24	0.21	-	1.35	0.04	0.05								
			Gr. 50	50	65	21	0.23	-	1.35	0.04	0.05								
Gr. 55			55	70		0.25	-	1.35	0.04	0.05									
Gr. 60	60		75	18	0.26	-	1.35	0.04	0.05										
A618	Gr. 65	65	80	17	0.26	-	1.35	0.04	0.05										
	Gr. I&II	50	70	22	0.2	0.2	1.35	0.04	0.05										
	Gr. III	50	65	22	.023	-	1.35	0.04	0.05			0.3							
A913	50	50	60	21	0.12	0.45	1.60	0.04	0.03	0.25	0.25	0.25	0.4	0.07	0.06				
	65	65	80	17	0.35	0.35	1.60	0.04	0.03	0.25	0.25	0.25	0.4	0.07	0.06				
Corrosion resistant, High-strength low-alloy	A992 ³	50–65	65	18	0.232	0.60	0.5– 0.15	0.35	0.45	0.45	0.45			0.15	0.11				
	A242	50	70	18	0.15	0.2	1.0	0.15	0.05										
	A588	50	70	21	0.19	0.2– 0.4	0.25– 1.35	0.04	0.05	0.04	0.05	0.4– 0.7			0.02– 0.1				

¹ Minimum unless range or other control noted.

² Two-inch gauge length.

³ A maximum-yield-to-tensile-strength ratio of 0.85 and carbon equivalent formula are included as mandatory in ASTM 992.

⁴ Several steel specifications can include a minimum copper content to provide weather resistance.

into a single specification, A7. The specification for A7 and A9 were limited to requirements for the tensile strength and yield point only; there were no chemical specifications. The chemical composition, particularly carbon content, became an issue during the 1950s, as welding gained favor for making structural connections. By 1964, AISC adopted five grades of steel for structural applications. The 1999 AISC Load and Resistance Factor Design Specification for Structural Steel Buildings, 1999, identifies 15 different ASTM steel designations for structural applications.

3.5.2 ■ Sectional Shapes

Figure 3.9 illustrates structural cross-sectional shapes commonly used in structural applications. These shapes are produced in different sizes and are designated with the letters W, HP, M, S, C, MC, and L. W shapes are doubly symmetric wide-flange shapes whose flanges are substantially parallel. HP shapes are also wide-flange shapes whose flanges and webs are of the same nominal thickness and whose depth and width are essentially the same. The S shapes are doubly symmetric shapes whose inside flange surfaces have approximately 16.67% slope. The M shapes are doubly symmetric shapes that cannot be classified as W, S, or HP shapes. C shapes are channels with inside flange surfaces having a slope of approximately 16.67%. MC shapes are channels that cannot be classified as C shapes. L shapes are angle shapes with either equal or unequal legs. In addition to these shapes, other structural sections are available, such as tee, sheet piling, and rail, as shown in Figure 3.9.

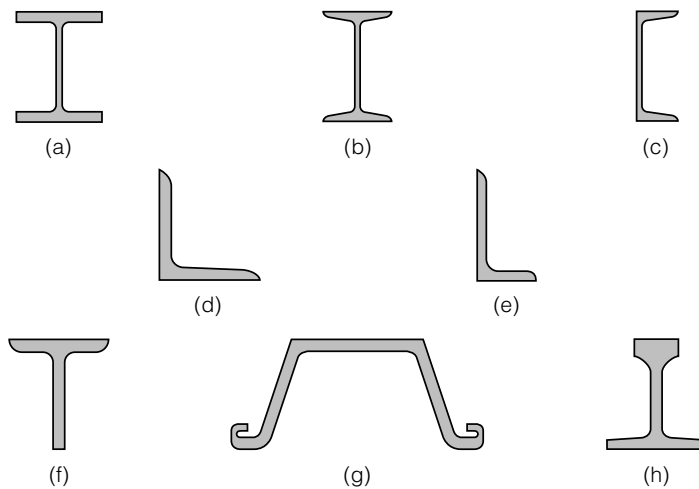


FIGURE 3.9 Shapes commonly used in structural applications: (a) wide-flange (W, HP, and M shapes), (b) I-beam (S shape), (c) channel (C and MC shapes), (d) equal-legs angle (L shape), (e) unequal-legs angle (L shape), (f) tee, (g) sheet piling, and (h) rail.

The W, M, S, HP, C, and MC shapes are designated by a letter, followed by two numbers separated by an \times . The letter indicates the shape, while the two numbers indicate the nominal depth and the weight per linear unit length. For example, W 44 \times 335 means W shape with a nominal depth of 44 in. and a weight of 335 lb/linear foot. An angle is designated with the letter L, followed by three numbers that indicate the leg dimensions and thickness in inches, such as L 4 \times 4 \times 1/2. Dimensions of these structural shapes are controlled by ASTM A6/A6M.

W shapes are commonly used as beams and columns, HP shapes are used as bearing piles, and S shapes are used as beams or girders. Composite sections can also be formed by welding different shapes to use in various structural applications. Sheet piling sections are connected to each other and are used as retaining walls.

Tables 3.3 and 3.4 summarize the applicable ASTM specifications/designations for structural steels, and plates and bars, respectively (Carter, 2004). These tables are guides only; specific information should be sought from the applicable specifications for each material and application. In particular, the dimension and application of a member can affect some finer points about material selection, which are not covered in these tables. In general, the materials identified as “preferred” in the tables are generally available in the market place. Those identified as “other applicable materials” may or may not be readily available.

3.5.3 ■ Specialty Steels in Structural Applications

As the ability to refine steels improves, it is possible to produce special products with sufficient economy to permit their use in construction projects. The Federal Highway Administration, US Navy, and AISI have taken a leading role in the development and application of high-performance steels. These are defined as materials that possess the optimum combination of properties required to build cost-effective structures that will be safe and durable throughout their service life (Lane et. al, 1998). One of the products developed through this effort is high-performance steels, HPS. Currently, two products are available: HPS 50W and HPS 70W. These are weathering steels that form a corrosion barrier on the surface of the steel when first exposed to the environment. This surface resists further corrosion, and hence reduces the need for maintenance. HPS 70W has stronger tensile properties than steel traditionally used for bridge construction, and hence bridges can be designed with a reduced quantity of material. These savings are somewhat offset by the cost of the material, but there is still a net reduction in construction costs. The tensile requirements of HPS 70W are yield strength 70,000 psi, tensile strength of 85,000 to 110,000 psi, and an elongation of 19% (2" gauge length). In addition, HPS must pass impact tests. HPS 70W is manufactured to tight and extensive alloy content requirements, as shown in Table 3.5 (ISG Plate, 2003).

Comparing the chemical requirements in Table 3.5 with those in Table 3.2 demonstrates that HPS 70W has more extensive chemical requirements,

TABLE 3.5 Chemical Requirements of HPS 70W

Element	Composition (% by weight)
Carbon	0.11 max
Manganese	1.10–1.35
Phosphorus	0.020 max
Sulfur*	0.006 max
Silicon	0.30–0.50
Copper	0.25–0.40
Nickel	0.25–0.40
Chromium	0.45–0.70
Molybdenum	0.02–0.08
Vanadium	0.04–0.08
Aluminum	0.01–0.04
Nitrogen	0.015 max

* All HPS 70 must be calcium treated for sulfide shape control

April 2002, more than 150 bridges with HPS have either been constructed or were in the design and construction stage (FHWA, 2002).

The desire to improve the appearance and durability of steel structures has produced an interest in designing structural members with stainless steel. The durability of stainless steel has long been recognized, but the cost of the material was prohibitive. The ability of stainless steel to resist corrosion rests in the high chromium content. Whereas common structural steels have 0.3 to 0.4 percent chromium, stainless steel has in excess of 10 percent, by definition. Five AISI grades of stainless steel are used for structural applications (SSINA, 1999):

- 304: the most readily available stainless steel, containing 18% chromium and 8% nickel. Excellent corrosion resistance and formability,
- 316: similar to 304, but with the addition of 3–4 percent molybdenum for greater corrosion resistance. Generally specified for highly corrosive environments such as industrial, chemical, and seacoast atmospheres
- 409: a straight chrome alloy, 11 to 12 percent chromium. Primarily used for interior applications.
- 410–3: a dual phase alloy with micro alloy element control that permits welding in up to 1.25 inches.
- 2205: a duplex structure with about equal parts of austenite and ferrite. Excellent corrosion resistance and about twice the yield strength of conventional grades.

The chemical and tensile properties of these grades are summarized in Table 3.6.

TABLE 3.6 Properties of Stainless Steels Used for Structural Applications

AISI Type	F _y (ksi)	F _u (ksi)	Percent Elong*	Components (typically maximum percent by weight)										
				C	Cr	Mn	Mo	N	Ni	P	S	Si	Ti	
304	31.2	73.2	70	0.08	18-20	2				8-10.5	0.045	0.03	1	
316	29.7	74.7	40	0.08	16-18	2					0.045	0.03	0.75	
409	34.8	65.3	25	0.08	11.13	1					0.045	0.045	1	0.75
410	178	221	45	0.15	12.5	1					0.04	0.03		
2205	74.7	110	35	0.02	22.4	0.7	3.3	0.16		5.8	0.25	0.001	0.4	

* Percent elongation is the percentage of plastic strain at fracture (2" gauge length)

3.6 Fastening Products

Fastening products include (Carter, 2004)

- Conventional bolts
- Twist-off-type tension control bolt assemblies
- Nuts
- Washers
- Compressible-washer-type direct tension indicators
- Anchor rods
- Threaded rods
- Forged steel structural hardware

Table 3.7 summarizes the applicable ASTM specifications for each type of fastener (Carter, 2004). High-strength bolts have a tensile strength in excess of 100,000 psi. Common bolts have a tensile strength of 60,000 psi. The preferred material for anchor rods, F1554 Grade 36, has a yield stress of 36,000 psi and an ultimate strength in the range of 58,000 to 80,000 psi. A36, with a yield stress of 36,000 psi, is preferred for threaded rods. Nuts, washers, and direct tension indicators are made with materials that do not have a minimum required strength.

Structural connections are made by riveting, bolting, or welding. Rivet connections were used extensively in the past, but modern bolt technology has made riveting obsolete. Bolted connections may be snug tightened, pretensioned, or slip critical (Miller, 2001). Snug-tightened joints are accomplished by either a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the members into firm contact. Pretensioned joints require tightening the bolt to a significant tensile stress with a corresponding compressive stress in the attached members. Four methods are used to ensure that the bolt is tightened to a sufficient stress level: turn-of-nut, calibrated wrench, twist-off-type tension-control bolts, and direct tension indicators. Bolts in slip-critical joints are also installed to pretensioned requirements, but these joints have “faying surfaces that have been prepared to provide a calculable resistance against slip.” When the joint is placed under load, the stresses may be transmitted through the joint by the friction between the members. However, if slip occurs, the bolts will be placed in shear, in addition to the tension stresses from the installation—hence the need for high-strength bolts.

3.7 Reinforcing Steel

Since concrete has negligible tensile strength, structural concrete members subjected to tensile and flexural stresses must be reinforced. Either conventional or prestressed reinforcing can be used, depending on the design

TABLE 3.7 Applicable ASTM Specifications for Structural Fasteners

ASTM Designation	F _y Yield Stress (ksi)	F _u Tensile Stress ^a (ksi)	Diameter Range (in)	High-strength Bolts	Common Bolts	Nuts	Washers	Direct Tension Indicators	Threaded Rods	Anchor Rods		
										Hooked	Headed	Threaded & Nutted
A325	-	105	>1–1.5	■								
	-	120	0.5–1.5									
A490	-	150	0.5–1.5									
F1852	-	105	1.125	■								
	-	120	0.5–1									
A194 Gr. 2H	-	-	0.25–4		■							
A563	-	-	0.25–4		■							
F436 ^b	-	-	0.25–4			■						
F959	-	-	0.5–1.5				■					
A36	36	58–80	<10					■				
A193 Gr. B7	-	100	>4–7						■			■
	-	115	>2.5–4							■		
	-	125	≤2.5									
A307	Gr. A	-	60	0.25–4		■						■
	Gr. C	-	58–80	0.25–4								
A354 Gr. BD	-	140	2.5–4									
		150	0.25–2.5									
A449	-	90	1.5–3		^c							
		105	1.125–1.5		^c							
		120	0.25–1		^c							
A572	Gr. 42	42	60	<6								
	Gr. 50	50	65	<4								
	Gr. 55	55	70	<2								
	Gr. 60	60	75	<1.25								
	Gr. 65	65	80	<1.25								
A588	42	63	>5–8									
	46	67	>4–5									
	50	70	<4									
A687	105	150 max	0.625–3									
F1554	Gr. 36	36	58–80	0.25–4								
	Gr. 55	55	75–95	0.25–4								
	Gr. 105	105	125–150	0.25–3								
	■	= Preferred material specification										
	■	= Other applicable material specification, the availability of which should be confirmed before specifying										
	□	= Material specification does not apply										
- indicates that a value is not specified in the material specification												
^a Minimum values unless range or max. is indicated												
^b Special washer requirements apply for some steel-to-steel bolting, check design documents												
^c LRFD has limitations on use of ASTM A449 bolts												

situation. In conventional reinforcing, the stresses fluctuate with loads on the structure. This does not place any special requirements on the steel. On the other hand, in prestressed reinforcement, the steel is under continuous tension. Any stress relaxation will reduce the effectiveness of the reinforcement. Hence, special steels are required.

Reinforcing steel (rebar) is manufactured in three forms: *plain bars*, *deformed bars*, and *plain and deformed wire fabrics*. Plain bars are round, without surface deformations. Plain bars provide only limited bond with the concrete and, therefore, are not typically used in sections subjected to tension or bending. Deformed bars have protrusions (deformations) at the surface, as shown Figure 3.10; thus, they ensure a good bond between the bar and the concrete. The deformed surface of the bar prevents slipping, allowing the concrete and steel to work as one unit. Wire fabrics are flat sheets in which wires pass each other at right angles, and one set of elements is parallel to the fabric axis. Plain wire fabrics develop the anchorage in concrete at the welded intersections, while deformed wire fabrics develop anchorage through deformations and at the welded intersections.

Deformed bars are used in concrete beams, slabs, columns, walls, footings, pavements, and other concrete structures, as well as in masonry construction. Welded wire fabrics are used in some concrete slabs and pavements, mostly to resist temperature and shrinkage stresses. Welded wire fabrics can be more economical to place, and thus allow for closer spacing of bars than is practical with individual bars.



FIGURE 3.10 Steel rebar used to reinforce PCC columns.

Reinforcing steel is produced in the standard sizes shown in Table 3.8. Bars are made of four types of steel: A615 (billet), A616 (rail), A617 (axle), and A706 (low-alloy), as shown in Table 3.9. Billet steel is the most widely used. A706 steel is often used when the rebar must be welded to structural steel. Reinforcing steel is produced in four grades: 40, 50, 60, and 75, with yield stresses of 276 MPa, 345 MPa, 414 MPa, and 517 MPa (40 ksi, 50 ksi, 60 ksi, and 75 ksi), respectively.

Prestressed concrete requires special wires, strands, cables, and bars. Steel for prestressed concrete reinforcement must have high strength and low relaxation properties. High-carbon steels and high-strength alloy steels are used for this purpose. Properties of prestressed concrete reinforcement are presented in ASTM specification A416 and AASHTO specification M203. These specifications define the requirements for a seven-wire uncoated steel strand. The specifications allow two types of steel: stress-relieved (normal-relaxation) and low-relaxation. Relaxation refers to the percent of stress reduction that occurs when a constant amount of strain is applied over an extended time period. Both stress-relieved and low-relaxation steels can be specified as Grade 250 or Grade 270, with ultimate strengths of 1725 MPa (250 ksi) and 1860 MPa (270 ksi), respectively. The specifications for this application are based on mechanical properties only; the chemistry of wires is not pertinent to this application. After stranding, low-relaxation strands are subjected to a continuous thermal–mechanical treatment to produce the required mechanical properties. Table 3.10 shows the required properties for seven-wire strand.

3.8 Mechanical Testing of Steel

Many tests are available to evaluate the mechanical properties of steel. This section summarizes some laboratory tests commonly used to determine properties required in product specifications. Test specimens can take several shapes, such as bar, tube, wire, flat section, and notched bar, depending on the test purpose and the application.

Certain methods of fabrication, such as bending, forming, and welding, or operations involving heating, may affect the properties of the material being tested. Therefore, the product specifications cover the stage of manufacture at which mechanical testing is performed. The properties shown by testing before the material is fabricated may not necessarily be representative of the product after it has been completely fabricated. In addition, flaws in the specimen or improper machining or preparation of the test specimen will give erroneous results (ASTM A370).

3.8.1 Tension Test

The tension test (ASTM E8) on steel is performed to determine the yield strength, yield point, ultimate (tensile) strength, elongation, and reduction

TABLE 3.8 Standard-Size Reinforcing Bars According to ASTM A615*

Bar Designation Number**	Nominal Mass (kg/m)	Diameter (mm)	Cross-Sectional Area (mm ²)	Perimeter (mm)	Deformation Requirements (mm)***		
					Maximum Average Spacing	Minimum Average Height	Maximum Gap****
10 [3]	0.560	9.5	71	29.9	6.7	0.38	3.6
13 [4]	0.994	12.7	129	39.9	8.9	0.51	4.9
16 [5]	1.552	15.9	199	49.9	11.1	0.71	6.1
19 [6]	2.235	19.1	284	59.8	13.3	0.97	7.3
22 [7]	3.042	22.2	387	69.8	15.5	1.12	8.5
25 [8]	3.973	25.4	510	79.8	17.8	1.27	9.7
29 [9]	5.059	28.7	645	90.0	20.1	1.42	10.9
32 [10]	6.404	32.3	819	101.3	22.6	1.63	12.4
36 [11]	7.907	35.8	1006	112.5	25.1	1.80	13.7
43 [14]	11.38	43.0	1452	135.1	30.1	2.16	16.5
57 [18]	20.24	57.3	2581	180.1	40.1	2.59	21.9

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** Bar numbers approximate the number of millimeters of the nominal diameter of the bars. [Bar numbers are based on the number of eighths of an inch of the nominal diameter of the bars.]

*** The nominal dimensions of a deformed bar are equivalent to those of a plain round bar having the same weight per meter as the deformed bar.

**** Requirements for protrusions on the surface of the bar.

***** Chord 12.5% of Nominal Perimeter

TABLE 3.9 Types and Properties of Reinforcing Bars According to ASTM (Somayaji, 2001) (Reprinted by permission of Pearson Education, Inc.)

ASTM Designation	Type	Grade	Tensile strength min., MPa (ksi)	Yield strength* Min., MPa (ksi)	Size availability (No.)
A615	Billet steel bars (plain and deformed)	40	483 (70)	276 (40)	3–6
		60	620 (90)	414 (60)	3–18
		75	689 (100)	517 (75)	11–18
A616	Rail steel (plain and deformed)	50	552 (80)	345 (50)	3–11
		60	620 (90)	474 (60)	3–11
A617	Axle steel (plain and deformed)	40	483 (70)	276 (40)	3–11
		60	620 (90)	414 (60)	3–11
A706	Low-alloy steel deformed bars	60	552 (80)	414–538 (60–78)	3–18

*When the steel does not have a well-defined yield point, yield strength is the stress corresponding to a strain of 0.005 m/m (0.5% extension) for grades 40, 50, and 60, and a strain of 0.0035 m/m (0.35% extension) for grade 75 of A615, A616, and A617 steels. For A706 steel, grade point is determined at a strain of 0.0035 m/m.

TABLE 3.10 Required Properties for Seven-Wire Strand

Property	Stress-relieved		Low-relaxation	
	Grade 250	Grade 270	Grade 250	Grade 270
Breaking strength,* MPa (ksi)	1725 (250)	1860 (270)	1725 (250)	1860 (270)
Yield strength (1% extension)	85% of breaking strength		90% of breaking strength	
Elongation (min. percent)	3.5		3.5	
Relaxation** (max. percent)				
Load = 70% min. breaking strength	—		2.5	
Load = 80% min. breaking strength	—		3.5	

*Breaking strength is the maximum load required to break one or more wires.

**Relaxation is the reduction in stress that occurs when a constant strain is applied over an extended time period. The specification is for a load duration of 1000 hours at a test temperature of $20 \pm 2^\circ\text{C}$ ($68 \pm 3^\circ\text{F}$).

of area. Typically, the test is performed at temperatures between 10°C and 35°C (50°F to 95°F).

The test specimen can be either full sized or machined into a shape, as prescribed in the product specifications for the material being tested. It is desirable to use a small cross-sectional area at the center portion of the specimen to ensure fracture within the gauge length. Several cross-sectional shapes are permitted, such as round and rectangular, as shown in Figure 3.11. Plate, sheet, round rod, wire, and tube specimens may be used. A 12.5 (1/2 in.) diameter round specimen is used in many cases. The gauge length over which the elongation is measured typically is four times the diameter for most round-rod specimens.

Various types of gripping devices may be used to hold the specimen, depending on its shape. In all cases, the axis of the test specimen should be placed at the center of the testing machine head to ensure axial tensile stresses within the gauge length without bending. An extensometer with a dial gauge (Figure 1.25) or an LVDT (Figure 1.29) is used to measure the deformation of the entire gauge length. The test is performed by applying an axial load to the specimen at a specified rate. Figure 3.12 shows a tensile test being performed on a round steel specimen using an LVDT extensometer to measure the deformation.

As discussed in Chapter 1, mild steel has a unique stress–strain relation (Figure 3.13). Here, a linear elastic response is displayed up to the proportion limit. As the stress is increased beyond the proportion limit, the steel will yield, at which time the strain will increase without an increase in stress (actually the stress will slightly decrease). As tension increases past the yield point, strain increases following a nonlinear relation up to the point of failure.

Note that the decrease in stress after the peak does not mean a decrease in strength. In fact, the actual stress continues to increase until failure. The reason for the apparent decrease is that a neck is formed in the steel specimen, causing an appreciable decrease in the cross-sectional area. The traditional, or engineering, way of calculating the stress and strain uses the original



FIGURE 3.11 Tension test specimens with round and rectangular cross-sections.



FIGURE 3.12 Tension test on a round steel specimen showing grips and an extensometer with an LVDT.

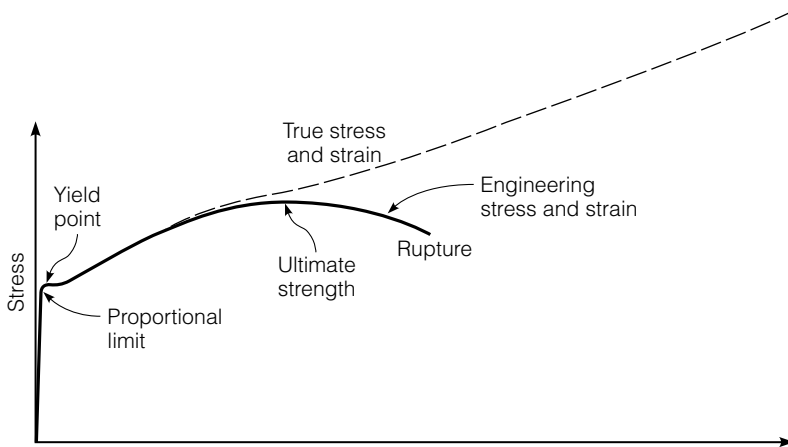


FIGURE 3.13 Typical stress-strain behavior of mild steel.

cross-sectional area and gauge length. If the stress and strains are calculated based on the instantaneous cross-sectional area and gauge length, a *true stress-strain curve* is obtained, which is different than the *engineering stress-strain curve* (Figure 3.13).

As shown in Figure 3.13, the true stress is larger than the engineering stress, because of the reduced cross-sectional area at the neck. Also, the true strain is larger than the engineering strain, since the increase in length at the vicinity of the neck is much larger than the increase in length outside of the neck. The specimen experiences the largest deformation (contraction of the cross-sectional area and increase in length) at the regions closest to the neck, due to the nonuniform distribution of the deformation.

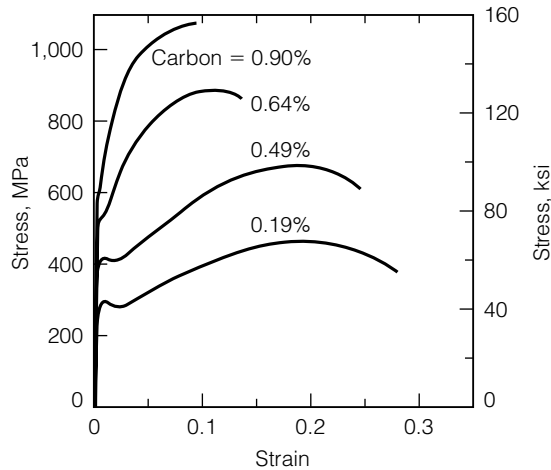


FIGURE 3.14 Tensile stress-strain diagrams of hot-rolled steel bars with different carbon contents.

The large increase in length at the neck increases the true strain to a large extent because the definition of true strain utilizes a ratio of the change in length in an infinitesimal gauge length. By decreasing the gauge length toward an infinitesimal size and increasing the length due to localization in the neck, the numerator of an expression is increased while the denominator stays small, resulting in a significant increase in the ratio of the two numbers. Note that when calculating the true strain, a small gauge length should be used at the neck, since the properties of the material (such as the cross section) at the neck represent the true material properties. For various practical applications, however, the engineering stresses and strains are used, rather than the true stresses and strains.

Different carbon-content steels have different stress-strain relations. Increasing the carbon content in the steel increases the yield stress and reduces the ductility. Figure 3.14 shows the tension stress-strain diagram for hot-rolled steel bars containing carbons from 0.19% to 0.90%. Increasing the carbon content from 0.19% to 0.90% increases the yield stress from 280 MPa to 620 MPa (40 ksi to 90 ksi). Also, this increase in carbon content decreases the fracture strain from about 0.27 m/m to 0.09 m/m. Note that the increase in carbon content does not change the modulus of elasticity.

Sample Problem 3.2

A steel alloy bar 100 mm long with a rectangular cross section of 10 mm \times 40 mm is subjected to tension with a load of 89 kN and experiences an increase in length of 0.1 mm. If the increase in length is entirely elastic, calculate the modulus of elasticity of the steel alloy.

Solution

$$\sigma = \frac{89000}{0.01 \times 0.04} = 0.225 \times 10^9 \text{ Pa} = 0.2225 \text{ GPa}$$

$$\varepsilon = \frac{0.1}{100} = 0.001 \text{ mm/mm}$$

$$E = \frac{\sigma}{\varepsilon} = \frac{0.2225}{0.001} = 222.5 \text{ GPa}$$

Sample Problem 3.3

A steel specimen is tested in tension. The specimen is 1 in. wide by 0.5 in. thick in the test region. By monitoring the load dial of the testing machine, it was found that the specimen yielded at a load of 36 kips and fractured at 48 kips.

- Determine the tensile stresses at yield and at fracture.
- If the original gauge length was 4 in., estimate the gauge length when the specimen is stressed to 1/2 the yield stress.

Solution

- Yield stress (σ_y) = $36/(1 \times 0.5) = 72$ ksi
Fracture stress (σ_f) = $48/(1 \times 0.5) = 96$ ksi

- Assume $E = 30 \times 10^6$ psi

$$\varepsilon = \left(\frac{1}{2}\right)\sigma_y/E = (1/2) \times 72 \times 10^3/(30 \times 10^6) = 0.0012 \text{ in./in.}$$

$$\Delta L = L\varepsilon = 4 \times 0.0012 = 0.0048 \text{ in.}$$

$$\text{Final gauge length} = 4 + 0.0048 = 4.0048 \text{ in.}$$

Steel is generally assumed to be a homogenous and isotropic material. However, in the production of structural members, the final shape may be obtained by cold rolling. This essentially causes the steel to undergo plastic deformations, with the degree of deformation varying throughout the member. As discussed in Chapter 1, plastic deformation causes an increase in yield strength and a reduction in ductility. Figure 3.15 demonstrates that the measured properties vary, depending on the orientation of the sample relative to the axis of rolling (Hassett, 2003). Thus, it is necessary to specify how the sample is collected when evaluating the mechanical properties of steel.

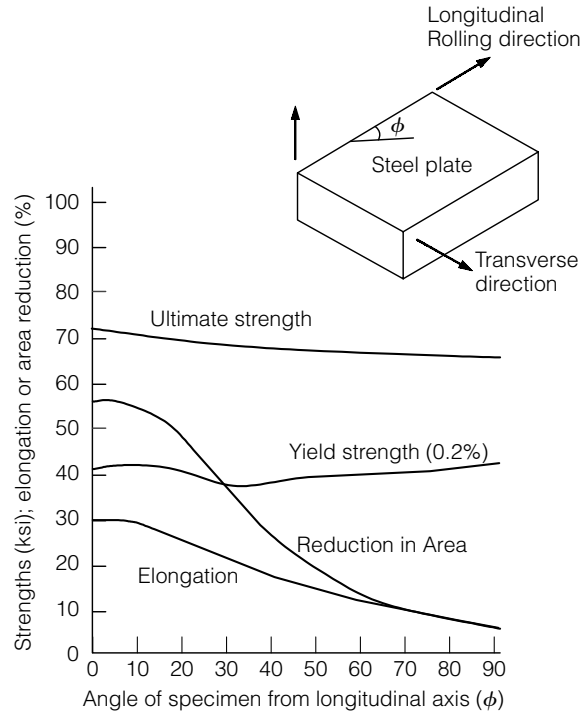


FIGURE 3.15 Example of effect of specimen orientation on measured tensile properties of steel.

3.8.2 Torsion Test

The torsion test (ASTM E143) is used to determine the shear modulus of structural materials. The shear modulus is used in the design of members subjected to torsion, such as rotating shafts and helical compression springs. In this test a cylindrical, or tubular, specimen is loaded either incrementally or continually by applying an external torque to cause a uniform twist within the gauge length (Figure 3.16). The amount of applied torque and the corresponding angle of twist are measured throughout the test. Figure 3.17 shows the shear stress–strain curve. The shear modulus is the ratio of maximum shear stress to the corresponding shear strain below the proportional limit of the material, which is the slope of the straight line between R (a pretorque stress) and P (the proportional limit). For a circular cross section, the maximum shear stress (τ_{\max}), shear strain (γ), and the shear modulus (G) are determined by the equations

$$\tau_{\max} = \frac{Tr}{J} \quad (3.1)$$

$$\gamma = \frac{\theta r}{L} \quad (3.2)$$

$$G = \frac{\tau_{\max}}{\gamma} = \frac{TL}{J\theta} \quad (3.3)$$

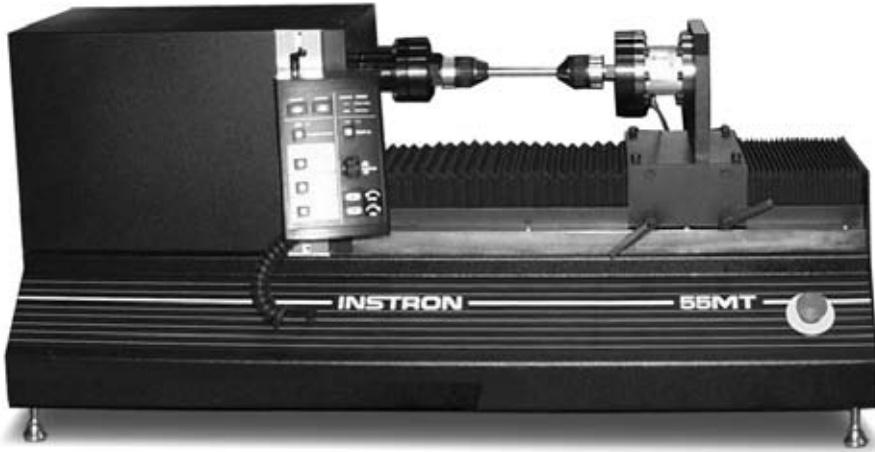


FIGURE 3.16 Torsion test apparatus.

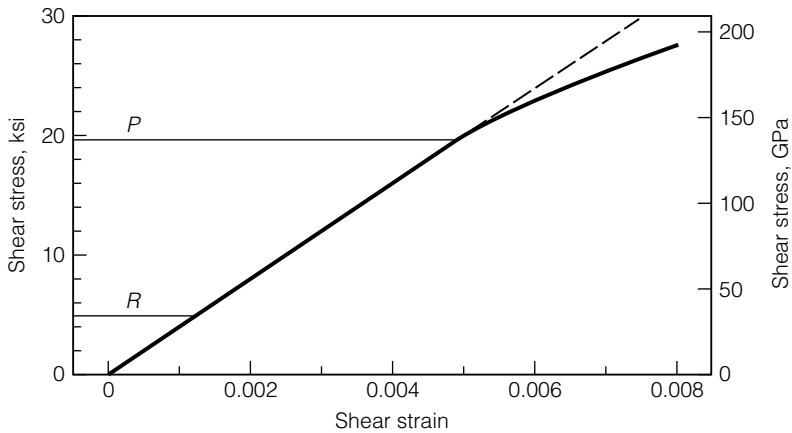


FIGURE 3.17 Typical shear stress-strain diagram of steel (ASTM E143). Copyright ASTM. Reprinted with permission.

where

T = torque

r = radius

J = polar moment of inertia of the specimen about its center, $\frac{\pi r^4}{2}$ for a solid circular cross section.

θ = angle of twist in radians

L = gauge length

The test method is limited to materials and stresses at which creep is negligible compared with the strain produced immediately upon loading. The

test specimen should be sound, without imperfections near the surface. Also, the specimen should be straight and of uniform diameter for a length equal to the gauge length plus two to four diameters. The gauge length should be at least four diameters. During the test, torque is read from a dial gauge or a readout device attached to the testing machine, while the angle of twist may be measured using a torsionmeter fastened to the specimen at the two ends of the gauge length. A curve-fitting procedure can be used to estimate the straight-line portion of the shear stress–strain relation of Figure 3.17 (ASTM E143).

Sample Problem 3.4

A rod with a length of 1 m and a radius of 20 mm is made of high-strength steel. The rod is subjected to a torque T , which produces a shear stress below the proportional limit. If the cross section at one end is rotated 45 degrees in relation to the other end, and the shear modulus G of the material is 90 GPa, what is the amount of applied torque?

Solution

$$J = \pi r^4/2 = \pi(0.02)^4/2 = 0.2513 \times 10^{-6} \text{ m}^4$$

$$\theta = 45(\pi/180) = \pi/4$$

$$\begin{aligned} T &= GJ\theta/L = (90 \times 10^9) \times (0.2513 \times 10^{-6}) \times (\pi/4)/1 \\ &= 17.8 \times 10^3 \text{ N}\cdot\text{m} = 17.8 \text{ kN}\cdot\text{m} \end{aligned}$$

3.8.3 Charpy V Notch Impact Test

The Charpy V Notch impact test (ASTM E23) is used to measure the toughness of the material or the energy required to fracture a V-notched simply supported specimen. The test is used for structural steels in tension members.

The standard specimen is $55 \times 10 \times 10 \text{ mm}$ ($2.165 \times 0.394 \times 0.394 \text{ in.}$) with a V notch at the center, as shown in Figure 3.18. Before testing, the specimen is brought to the specified temperature for a minimum of 5 min in a liquid bath or 30 min in a gas medium. The specimen is inserted into the Charpy V notch impact-testing machine (Figure 3.19) using centering tongs. The swinging arm of the machine has a striking tip that impacts the specimen on the side opposite the V notch. The striking head is released from the pretest position, striking and fracturing the specimen. By fracturing the test specimen, some of the kinetic energy of the striking head is absorbed, thereby reducing the ultimate height the strike head attains. By measuring the height the strike head attains after striking the specimen,



FIGURE 3.18 Charpy V notch specimens.



FIGURE 3.19 Charpy V notch impact testing machine.

the energy required to fracture the specimen is computed. This energy is measured in $\text{m} \cdot \text{N}$ ($\text{ft} \cdot \text{lb}$) as indicated on a gauge attached to the machine.

The lateral expansion of the specimen is typically measured after the test using a dial gauge device. The lateral expansion is a measure of the plastic deformation during the test. The higher the toughness of the steel, the larger the lateral expansion.

Figure 1.13, in Chapter 1, shows the typical energy (toughness) required to fracture structural steel specimens at different temperatures. The figure

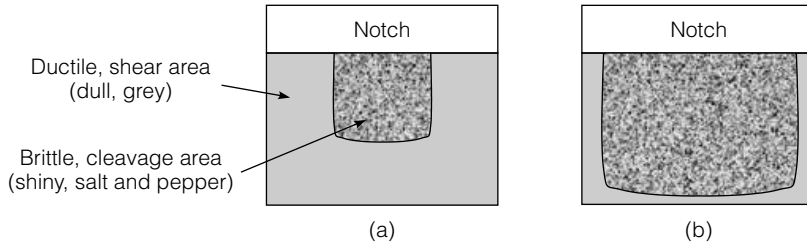


FIGURE 3.20 Fracture surface of Charpy V notch specimen: (a) at high temperature and (b) at low temperature.

shows that the required energy is high at high temperatures and low at low temperatures. This indicates that the material changes from ductile to brittle as the temperature decreases.

The fracture surface typically consists of a dull shear area (ductile) at the edges and a shiny cleavage area (brittle) at the center, as depicted in Figure 3.20. As the toughness of the steel decreases, due to lowering the temperature, for example, the shear area decreases while the cleavage area increases.

Sample Problem 3.5

A Charpy V Notch (CVN) test was performed on a steel specimen and produced the following readings:

Temperature (°F)	Toughness (ft.lb)
-40	5
30	7
100	28
170	66
240	79
310	80

Plot the toughness-versus-temperature relation, and determine the temperature transition zone between ductile and brittle behavior.

Solution

The toughness-versus-temperature relation is as shown in Figure SP3.5. From the figure, the temperature transition zone between ductile and brittle behavior can be seen to be 30 to 240°F.

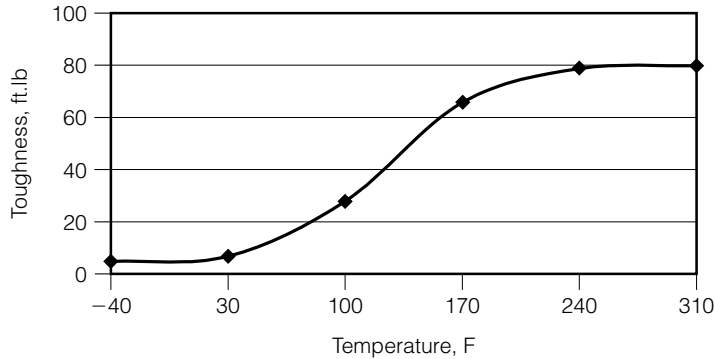


FIGURE SP3.5

3.8.4 Bend Test

In many engineering applications, steel is bent to a desired shape, especially in the case of reinforcing steel. The ductility to accommodate bending is checked by performing the semiguided bend test (ASTM E290). The test evaluates the ability of steel, or a weld, to resist cracking during bending. The test is conducted by bending the specimen through a specified angle and to a specified inside radius of curvature. When complete fracture does not occur, the criterion for failure is the number and size of cracks found on the tension surface of the specimen after bending.

The bend test is made by applying a transverse force to the specimen in the portion that is being bent, usually at midlength. Three arrangements can be used, as illustrated in Figure 3.21. In the first arrangement, the specimen is fixed at one end and bent around a reaction pin or mandrel by applying a force near the free end, as shown in Figure 3.21(a). In the second arrangement, the specimen is held at one end and a rotating device is used to bend the specimen around the pin or mandrel, as shown in Figure 3.21(b). In the third arrangement, a force is applied in the middle of a specimen simply supported at both ends, Figure 3.21(c).

3.8.5 Hardness Test

Hardness is a measure of a material's resistance to localized plastic deformation, such as a small dent or scratch on the surface of the material. A certain hardness is required for many machine parts and tools. Several tests are available to evaluate the hardness of materials. In these tests an indenter (penetrator) is forced into the surface of the material with a specified load

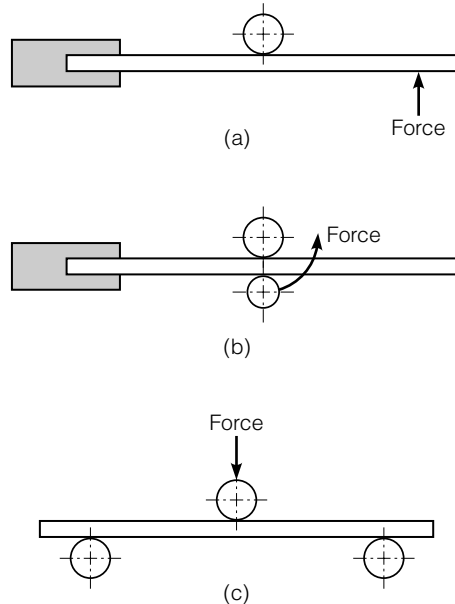


FIGURE 3.21 Schematic fixtures for semi-guided bend test (ASTM E290). Copyright ASTM. Reprinted with permission.

magnitude and rate of application. The depth, or the size, of the indentation is measured and related to a hardness index number. Hard materials result in small impressions, corresponding to high hardness numbers. Hardness measurements depend on test conditions and are, therefore, relative. Correlations and tables are available to convert the hardness measurements from one test to another and to approximate the tensile strength of the material (ASTM A370).

One of the methods commonly used to measure hardness of steel and other metals is the *Rockwell hardness test* (ASTM E18). In this test the depth of penetration of a diamond cone, or a steel ball, into the specimen is determined under fixed conditions (Figure 3.22). A preliminary load of 10 kg is applied first, followed by an additional load. The Rockwell number, which is proportional to the difference in penetration between the preliminary and total loads, is read from the machine by means of a dial, digital display, pointer, or other device. Two scales are frequently used, namely, B and C. Scale B uses a 1.588 mm (1/16 in.) steel ball indenter and a total load of 100 kg, while scale C uses a diamond spheroconical indenter with a 120° angle and a total load of 150 kg.

To test very thin steel or thin surface layers, the *Rockwell superficial hardness test* is used. The procedure is the same as the Rockwell hardness test except that smaller preliminary and total loads are used. The Rockwell hardness number is reported as a number, followed by the symbol HR, and another symbol representing the indenter and forces used. For example, 68 HRC indicates a Rockwell hardness number of 68 on Rockwell C scale.



FIGURE 3.22 Rockwell hardness test machine.

Hardness tests are simple, inexpensive, nondestructive, and do not require special specimens. In addition, other mechanical properties, such as the tensile strength, can be estimated from the hardness numbers. Therefore, hardness tests are very common and are typically performed more frequently than other mechanical tests.

3.8.6 ■ Ultrasonic Testing

Ultrasonic testing is a nondestructive method for detecting flaws in materials. It is particularly useful for the evaluation of welds. During the test, a sound wave is directed toward the weld joint and reflected back from a discontinuity, as shown on Figure 3.23. A sensor captures the energy of the reflected wave and the results are displayed on an oscilloscope. This method is highly sensitive in detecting planar defects, such as incomplete weld fusion, delamination, or cracks (Hassett, 2003).

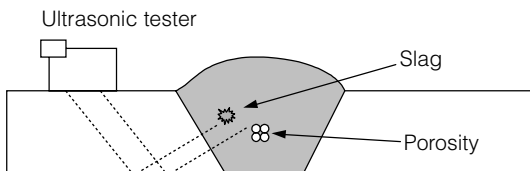


FIGURE 3.23 Ultrasonic test of welds.

3.9 Welding

Many civil engineering structures, such as steel bridges, frames, and trusses require welding during construction and repair. Welding is a technique for joining two metal pieces by applying heat to fuse the pieces together. A filler metal may be used to facilitate the process. The chemical properties of the welding material must be carefully selected to be compatible with the materials being welded. A variety of welding methods are available, but the common types are arc welding and gas welding. Other types of welding include flux-cored arc welding, self-shielded flux arc welding, and electroslag welding (Frank and Smith 1990).

Arc welding uses an arc between the electrode and the grounded base metal to bring both the base metal and the electrode to their melting points. The resulting deposited weld metal is a cast structure with a composition dependent upon the base metal, electrode, and flux chemistry. *Shielded metal arc welding (stick welding)* is the most common form of arc welding. It is limited to short welds in bridge construction. A consumable electrode, which is covered with flux, is used. The flux produces a shielding atmosphere at the arc to prevent oxidation of the molten metal. The flux is also used to trap impurities in the molten weld pool. The solidified flux forms a slag that covers the solidified weld, as shown in Figure 3.24. *Submerged arc welding* is a semiautomatic or automatic arc welding process. In this process, a bare wire electrode is automatically fed by the welding machine while a granular flux is fed into the joint ahead of the electrode. The arc takes place in the molten flux, which completely shields the weld pool from the atmosphere. The molten flux concentrates the arc heat, resulting in deep penetration into the base metal.

Gas welding (mig welding) is another type of welding in which no flux is used. An external shielding gas is used, which shields the molten weld

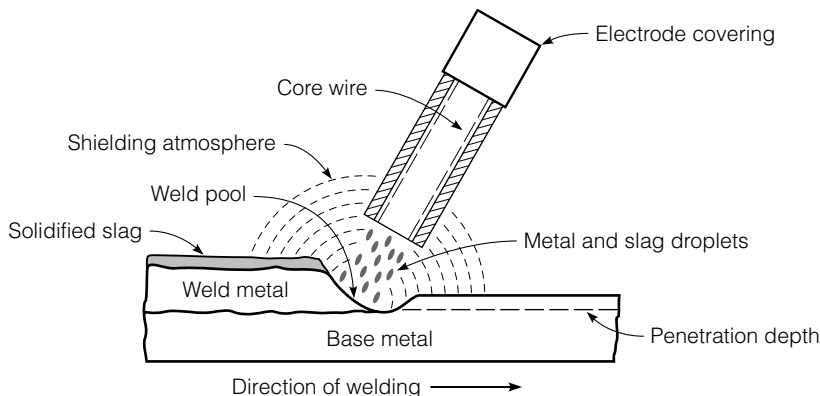


FIGURE 3.24 Schematic drawing of arc welding.

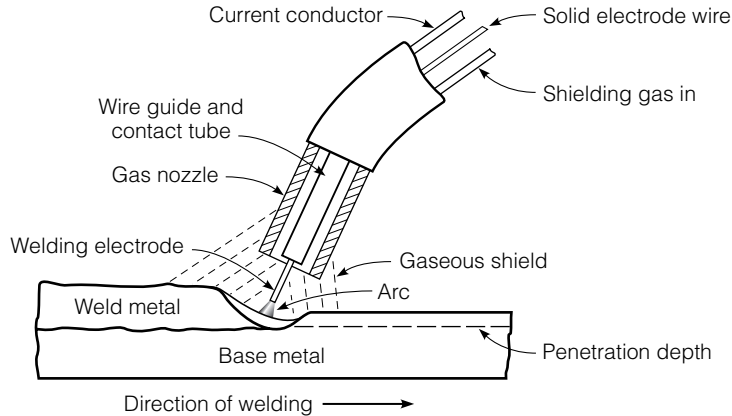


FIGURE 3.25 Schematic drawing of gas welding.

pool and provides the desired arc characteristics (Figure 3.25). This welding process is normally used for small welds due to the lack of slag formation.

Care must be taken during welding to consider the distortion that is the result of the nonuniform heating of the welding process. When the molten weld metal cools, it shrinks, causing deformation of the material and introducing residual stresses into the structure. Frequently, these residual stresses cause cracks outside the weld area. The distortion produced by welding can be controlled by proper sequencing of the welds and preforming the components prior to welding. Finally, care must be taken by the welder and the inspector to protect their eyes and skin from the intensive ultraviolet radiation produced during welding (Frank and Smith 1990).

The relative ease with which steel can be welded is related to the hardness of the steel. In general, harder steels are more difficult to weld. Winter-ton developed the concept of using a carbon equivalent formula for estimating the carbon equivalent of steels and an associated zone chart for determining the need to preheat steel to control the development of hydrogen in the welded steel. There are several different carbon equivalent formulas. The one used for structural steels is

$$CE = C + \frac{(Mn + Si)}{6} + \frac{(Cr + Mo + V)}{5} + \frac{(Ni + Cu)}{15} \quad (3.4)$$

Figure 3.26 shows the zones associated with the different combinations of carbon and carbon equivalent. The zones are used as a guide to determining the method used to determine preheat requirements (Hassett, 2003):

- Zone I—Cracking is unlikely, but may occur with high hydrogen or high restraint. Use hydrogen control method to determine preheat.
- Zone II—The hardness control method and selected hardness shall be used to determine minimum energy input for single-pass fillet welds without preheat.
- Zone III—The hydrogen control method shall be used.

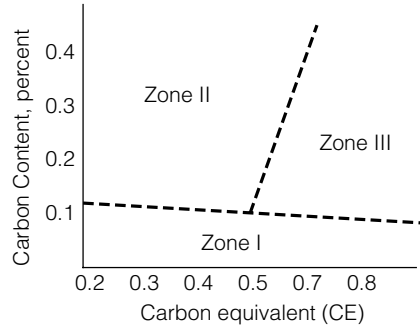


FIGURE 3.26 Welding zone classification of steel.

The hardness and hydrogen control methods are means of determining the level of energy used to preheat the weld area before the weld is performed. Preheating of the metal and the use of low hydrogen electrodes are the best means of avoiding hydrogen embrittlement. This occurs when steel is melted during welding, which may allow hydrogen to dissolve in the molten metal and diffuse into the base metal adjacent to the weld.

Whenever metal is welded, the base material adjacent to the weld is heated to a temperature that may be sufficient to affect its metallurgy. The material affected in this manner is termed the heat-affected zone, HAZ (Figure 3.27). The material in the HAZ is a high-risk area for failure, especially if proper preheating and cooling procedures are not followed.

3.10 Steel Corrosion

Corrosion is defined as the destruction of a material by electrochemical reaction to the environment. For simplicity, corrosion of steel can be defined as the destruction that can be detected by rust formation. Corrosion of steel

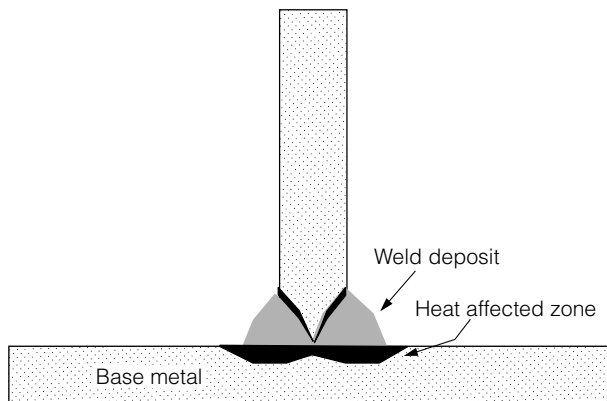


FIGURE 3.27 Heat affected zone produced during welding.

structures can cause serious problems and embarrassing and/or dangerous failures. For example, corrosion of steel bridges, if left unchecked, may result in lowering weight limits, costly steel replacement, or collapse of the structure. Other examples include corrosion of steel pipes, trusses, frames, and other structures. It is estimated that the cost of corrosion in the United States alone is \$8 billion each year (Frank and Smith 1990).

Corrosion is an electrochemical process; that is, it is a chemical reaction in which there is transfer of electrons from one chemical species to another. In the case of steel, the transfer is between iron and oxygen, a process called *oxidation reduction*. Corrosion requires the following four elements (without any of them corrosion will not occur):

1. an *anode*—the electrode where corrosion occurs
2. a *cathode*—the other electrode needed to form a corrosion cell
3. a *conductor*—a metallic pathway for electrons to flow
4. an *electrolyte*—a liquid that can support the flow of electrons

Steel, being a heterogeneous material, contains anodes and cathodes. Steel is also an electrical conductor. Therefore, steel contains three of the four elements needed for corrosion, while moisture is usually the fourth element (electrolyte).

The actual electrochemical reactions that occur when steel corrodes are very complex. However, the basic reactions for atmospherically exposed steel in a chemically neutral environment are dissolution of the metal at the anode and reduction of oxygen at the cathode.

Contaminants deposited on the steel surface affect the corrosion reactions and the rate of corrosion. Salt, from deicing or a marine environment, is a common contaminant that accelerates corrosion of steel bridges and reinforcing steel in concrete.

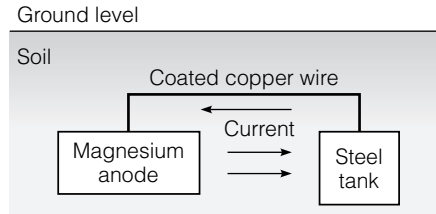
The environment plays an important role in determining corrosion rates. Since an electrolyte is needed in the corrosion reaction, the amount of time the steel stays wet will affect the rate of corrosion. Also, contaminants in the air, such as oxides or sulfur, accelerate corrosion. Thus, areas with acid rain, coal-burning power plants, and other chemical plants may accelerate corrosion.

3.10.1 ■ Methods for Corrosion Resistance

Since steel contains three of the four elements needed for corrosion, protective coatings can be used to isolate the steel from moisture, the fourth element. There are three mechanisms by which coatings provide corrosion protection (Hare 1987):

1. *Barrier coatings* work solely by isolating the steel from the moisture. These coatings have low water and oxygen permeability.
2. *Inhabitive primer coatings* contain passivating pigments. They are low-solubility pigments that migrate to the steel surface when moisture passes through the film to passivate the steel surface.

FIGURE 3.28 Cathodic protection of an underground pipeline using a magnesium sacrificial anode.



3. *Sacrificial primers (cathodic protection)* contain pigments such as elemental zinc. Since zinc is higher than iron in the galvanic series, when corrosion conditions exist the zinc gives up electrons to the steel, becomes the anode, and corrodes to protect the steel. There should be close contact between the steel and the sacrificial primer in order to have an effective corrosion protection.

Cathodic protection can take forms other than coating. For example, steel structures such as water heaters, underground tanks and pipes, and marine equipment, can be electrically connected to another metal that is more reactive in the particular environment, such as magnesium or zinc. Such reactive metal (sacrificial anode) experiences oxidation and gives up electrons to the steel, protecting the steel from corrosion. Figure 3.28 illustrates an underground steel tank that is electrically connected to a magnesium sacrificial anode (Fontana and Green 1978).

S U M M A R Y

The history of civil engineering is closely tied to that of steel, and this will continue into the foreseeable future. With the development of modern production facilities, the availability of a wide variety of economical steel products is virtually assured. High strength, ductility, the ability to carry tensile as well as compressive loads, and the ability to join members either with welding or mechanical fastening are the primary positive attributes of steel as a structural material. The properties of steel can be tailored to meet the needs of specific applications through alloying and heat treatments. The primary shortcoming of steel is its tendency to corrode. When using steel in structures, the engineer should consider the means for protecting the steel from corrosion over the life of the structure.

Q U E S T I O N S A N D P R O B L E M S

- 3.1 What is the chemical composition of steel? What is the effect of carbon on the mechanical properties of steel?
- 3.2 Why does the iron–carbon phase diagram go only to 6.7% carbon?
- 3.3 Draw a simple iron–carbon phase diagram showing the liquid, liquid–solid, and solid phases.
- 3.4 What is the typical maximum percent of carbon in steel used for structures?
- 3.5 Calculate the amounts and compositions of phases and constituents of steel composed of iron and 0.10% carbon just above and below the eutectoid isotherm.
- 3.6 Briefly discuss four heat treatment methods to enhance the properties of steel. What are the advantages of each treatment?
- 3.7 Define alloy steels. Explain why alloys are added to steel.
- 3.8 Name three alloying agents and their principal effects.
- 3.9 Specifically state the shape and size of the structural steel section: W 36 × 182.
- 3.10 What are the typical uses of structural steel?
- 3.11 Why is reinforcing steel used in concrete? Discuss the typical properties of reinforcing steel.
- 3.12 What is high performance steel? State two HPS products that are currently being used in structural applications and show their properties.
- 3.13 Name three mechanical tests used to measure properties of steel.
- 3.14 The following laboratory tests are performed on steel specimens:
 - a. Tension test
 - b. Charpy V notch test
 - c. Bend testWhat are the significance and use of these tests?
- 3.15 Sketch the stress–strain behavior of steel, and identify different levels of strength. What is the effect of increasing the carbon content in steel? What is a typical value for yield strength of mild steel?
- 3.16 Draw a typical stress–strain relationship for steel subjected to tension. On the graph, show the modulus of elasticity, the yield strength, the ultimate stress, and the rupture stress.
- 3.17 A steel specimen is tested in tension. The specimen is 1.5 in. wide by 0.5 in. thick in the test region. By monitoring the load dial of the

testing machine, it was found that the specimen yielded at a load of 37.5 kips and fractured at 52.5 kips.

- a. Determine the tensile stresses at yield and at fracture.
 - b. Estimate how much increase in length would occur at 60% of the yield stress in a 2-in. gauge length.
- 3.18 A round steel alloy bar with a diameter of 0.5 in. and a gauge length of 3.2 in. was subjected to tension, with the results shown in the accompanying table. Using a computer spreadsheet program, plot the stress–strain relationship. From the graph, determine the Young’s modulus of the steel alloy and the deformation corresponding to a 8225-lb load.

Load, lb	Deformation, 10^{-4} in.
2,000	11.28
4,000	22.54
6,000	33.80
8,000	45.08
10,000	56.36
12,000	67.66

- 3.19 Testing a round steel alloy bar with a diameter of 15 mm and a gauge length of 250 mm produced the stress–strain relation shown in Figure P3.19.

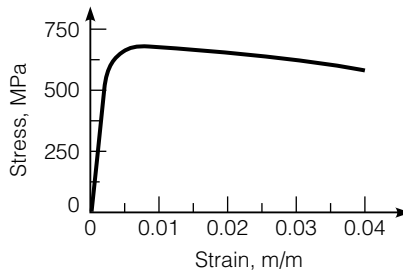


FIGURE P3.19

Determine

- a. the elastic modulus
- b. the proportional limit
- c. the yield strength at a strain offset of 0.002
- d. the tensile strength
- e. the magnitude of the load required to produce an increase in length of 0.38 mm
- f. the final deformation, if the specimen is unloaded after being strained by the amount specified in (e)

In designing a typical structure made of this material, would you expect the stress applied in (e) reasonable? Why?

3.20 During the tension test on a steel rod within the elastic region the following data were measured:

Applied load	= 102 kN
Original diameter	= 25 mm
Current diameter	= 24.99325 mm
Original gauge length	= 100 mm
Current gauge length	= 100.1 mm

Calculate the Young's modulus and Poisson's ratio.

3.21 A grade 36 round steel bar with a diameter of 0.5 inches and a gauge length of 2 inches was subjected to tension to rupture following ASTM E-8 test procedure. The load and deformation data were as shown in the following table:

Load (kips)	Displacement (in.)	Load (kips)	Displacement (in.)
0	0	8.56	0.08301
2.75	0.00096	8.79	0.09557
4.07	0.00141	8.98	0.10878
7.12	0.00242	9.15	0.12207
7.14	0.01691	9.25	0.13372
7.34	0.04196	9.35	0.14741
7.53	0.04599	9.44	0.18199
7.91	0.05847	7.87	0.29814
8.28	0.07117		

Using a spreadsheet program obtain the following:

- A plot of the stress–strain relationship. Label the axes and show units.
- A plot of the linear portion of the stress–strain relationship. Determine modulus of elasticity using the best fit approach.
- Proportional limit.
- Yield stress.
- Ultimate strength.
- When the applied load was 4.07 kips, the diameter was measured as 0.499905 inches. Determine Poisson's ratio.
- After the rod was broken, the two parts were put together and the diameter at the neck was measured as 0.416012 in. What is the true stress value at fracture? Is the true stress at fracture larger or smaller than the engineering stress at fracture? Why?
- Do you expect the true strain at fracture to be larger or smaller than the engineering strain at fracture? Why?

3.22 A high-yield-strength alloy steel bar with a rectangular cross section that has a width of 37.5 mm, a thickness of 6.25 mm, and a gauge length of 203 mm was tested in tension to rupture, according to ASTM E-8 method. The load and deformation data were as shown in Table P3.22.

Table P3.22

Load (kN)	Displacement (mm)	Load (kN)	Displacement (mm)
0	0	159.4	8.560
18.5	0.102	161.2	9.856
84.3	0.379	162.8	11.217
147.4	0.623	164.2	12.588
147.6	1.744	165.0	13.789
149.2	4.327	165.8	15.201
150.8	4.742	166.6	18.767
153.9	6.030	153.6	30.746
157.0	7.339		

Using a spreadsheet program, obtain the following:

- A plot of the stress–strain relationship. Label the axes and show units.
 - A plot of the linear portion of the stress–strain relationship. Determine modulus of elasticity using the best-fit approach.
 - Proportional limit.
 - Yield stress.
 - Ultimate strength.
 - If the specimen is loaded to 155 kN only and then unloaded, what is the permanent deformation?
 - In designing a typical structure made of this material, would you expect the stress applied in (f) safe? Why?
- 3.23 An ASTM A615 grade 60 number 10 rebar with a gauge length of 8 inches was subjected to tension to fracture according to ASTM E-8 method. The load and deformation data were as shown Table P3.23.

Table P3.23

Load (kips)	Displacement (in.)	Load (kips)	Displacement (in.)
0	0	106.27	0.3320
13.97	0.0036	109.46	0.3823
42.46	0.0094	111.55	0.4351
74.11	0.0163	113.64	0.4883
86.15	0.0676	115.23	0.5392
95.72	0.1592	117.44	0.6032
100.51	0.2339	119.65	0.7279
103.94	0.2847	118.18	0.8832

Using a spreadsheet program obtain the following:

- a. A plot of the stress–strain relationship. Label the axes and show units.
 - b. A plot of the linear portion of the stress–strain relationship. Determine modulus of elasticity using the best-fit approach.
 - c. Proportional limit.
 - d. Yield stress.
 - e. Ultimate strength.
 - f. If the rebar is loaded to 88,000 lb only and then unloaded, what is the permanent change in length?
- 3.24 A steel pipe having a length of 1 m, an outside diameter of 0.2 m, and a wall thickness of 10 mm, is subjected to an axial compression of 200 kN. Assuming a modulus of elasticity of 200 GPa and a Poisson’s ratio of 0.3, find
- a. the shortening of the pipe,
 - b. the increase in the outside diameter, and
 - c. the increase in the wall thickness.
- 3.25 A drill rod with a diameter of 12 mm is made of high-strength steel alloy with a shear modulus of 80 GPa. The rod is to be subjected to a torque T . What is the minimum required length L of the rod so that the cross section at one end can be rotated 90° with respect to the other end without exceeding an allowable shear stress of 300 MPa?
- 3.26 What is the shear modulus of the material whose shear stress–strain relation is shown in Figure 3.17? Solve the problem using
- a. SI units
 - b. U.S. customary units
- 3.27 An engineering technician performed a tension test on an A36 mild steel specimen to fracture. The original diameter of the specimen is 0.5 in. and the gauge length is 2.0 in. The information obtained from this experiment consists of applied tensile load (P) and increase in length (ΔL). The results are tabulated in Table P3.27. Using a spreadsheet program, complete the table by calculating the engineering stress (σ) and the engineering strain (ε). Determine the toughness of the material (u_t) by calculating the area under the stress–strain curve, namely,

$$u_t = \int_0^{\varepsilon_f} \sigma \, d\varepsilon$$

where ε_f is the strain at fracture. The preceding integral can be approximated numerically using a trapezoidal integration technique:

$$u_t = \sum_{i=1}^n u_i = \sum_{i=1}^n \frac{1}{2} (\sigma_i + \sigma_{i-1}) (\varepsilon_i - \varepsilon_{i-1})$$

- 3.28 A Charpy V Notch (CVN) test was performed on a steel specimen and produced the readings shown in Table P3.28. Plot the toughness-versus-temperature relation and determine the temperature transition zone between ductile and brittle behavior.

Table P3.27

Observation No.	P (lb)	ΔL (in.)	σ (psi)	ε (in./in.)	u_i (psi)
0	0	0.0000			N/A
1	1000	0.0005			
2	2730	0.0010			
3	4180	0.0015			
4	6360	0.0020			
5	8220	0.0025			
6	9310	0.0030			
7	9310	0.0035			
8	9310	0.0040			
9	9320	0.0045			
10	9330	0.0050			
11	9420	0.0100			
12	9450	0.0150			
13	9460	0.0200			
14	9510	0.0250			
15	9690	0.0300			
16	9560	0.0350			
17	9540	0.0400			
18	9650	0.0450			
19	10,060	0.0500			
20	11,870	0.1000			
21	12,830	0.1500			
22	13,360	0.2000			
23	13,670	0.2500			
24	13,850	0.3000			
25	13,920	0.3500			
26	13,960	0.4000			
27	13,800	0.5000			
28	13,600	0.5500			
29	13,150	0.6000			
30	12,510	0.6500			
31	11,690	0.7000			

$u_i =$

Table P3.28

Temperature, C	Toughness, Joule (N.m)
-40	7
0	15
50	54
100	109
150	120
200	122

3.29 A Charpy V Notch test was conducted for an ASTM A572 Grade 50 bridge steel. The average values of the test results at four different test temperatures were found to be

- 10 ft.lb at -50°F
- 15 ft.lb at 0°F
- 40 ft.lb at 40°F
- 60 ft.lb at 100°F

The bridge will be located in a region where specifications require a minimum of 25 ft.lb fracture toughness at 30°F for welded fracture-critical members. If the bridge contains a welded flange in a fracture-critical member, does the steel have adequate Charpy V notch fracture toughness? Show your supporting calculations.

- 3.30 How can the flaws in steel and welds be detected? Discuss the concept of a nondestructive test used for this purpose.
- 3.31 Determine the welding zone classification of A36 and A922 steel.
- 3.32 Briefly define steel corrosion. What are the four elements necessary for corrosion to occur?
- 3.33 Discuss the main methods used to protect steel from corrosion.

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4

ALUMINUM

Aluminum is the most plentiful metal on Earth, representing 8% of its crust. Although plentiful, aluminum exists primarily as oxides. The process of extracting aluminum from oxide is very energy intensive. In fact, approximately 2% to 3% of the electricity used in the United States is consumed in aluminum production. This high energy requirement makes recycling of aluminum products economical. Of the 24 million tons of aluminum produced annually, approximately 75% is from ore reduction and 25% is from recycled materials.

The properties of pure aluminum are not suitable for structural applications. Some industrial applications require pure aluminum, but otherwise, alloying elements are almost always added. These alloying elements, along with cold working and heat treatments, impart characteristics to the aluminum that make this product suitable for a wide range of applications. Figures 4.1 and 4.2 show aluminum used in two construction applications. Here, the term *aluminum* is used to refer to both the pure element and to alloys.

In terms of the amount of metal produced, aluminum is second only to steel. About 25% of aluminum produced is used for containers and packaging, 20% for architectural applications, such as doors, windows, and siding, and 10% for electrical conductors. The balance is used for industrial goods, consumer products, aircraft, and highway vehicles.

Aluminum accounts for 80% of the structural weight of aircraft, and its use in the automobile and light truck industry has increased 300% since 1971 (Reynolds Metals Company 1996). However, use of aluminum for infrastructure applications has been limited. Of the approximately 600,000 bridges in the United States, only nine have primary structural members made of aluminum. Two reasons for the limited use of aluminum are the relatively high initial cost when compared with steel and the lack of performance information on aluminum structures.



FIGURE 4.1 Aluminum frame used for structural support of a building.



FIGURE 4.2 Building facade made of aluminum.

Aluminum has many favorable characteristics and a wide variety of applications. The advantages of aluminum are that it (Budinski 1996)

- has one-third the density of steel
- has good thermal and electrical conductivity
- has high strength-to-weight ratio
- can be given a hard surface by anodizing and hard coating
- has alloys that are weldable
- will not rust
- has high reflectivity
- can be die cast
- is easily machined
- has good formability
- is nonmagnetic
- is nontoxic

Aluminum's high strength-to-weight ratio and its ability to resist corrosion are the primary factors that make aluminum an attractive structural engineering material. Although aluminum alloys can be formulated with strengths similar to steel products, the modulus of elasticity of aluminum is only about one-third that of steel. Thus, the dimensions of structural elements must be increased to compensate for the lower modulus of elasticity of aluminum.

4.1 Aluminum Production

Aluminum production uses processes that were developed in the 1880s. Bayer developed the sodium aluminate leaching process to produce pure alumina (Al_2O_3). Hall and Héroult, working independently, developed an electrolytic process for reducing the alumina to pure aluminum. The essence of the aluminum production process is shown in Figure 4.3.

The production of aluminum starts with the mining of the aluminum ore, bauxite. Commercial grade bauxite contains between 45% and 60% alumina. The bauxite is crushed, washed to remove clay and silica materials, and is kiln dried to remove most of the water. The crushed bauxite is mixed with soda ash and lime and passed through a digester, pressure reducer, and settling tank to produce a concentrated solution of sodium aluminate. This step removes silica, iron oxide, and other impurities from the sodium aluminate solution. The solution is seeded with hydrated alumina crystals in precipitator towers. The seeds attract other alumina crystals and form groups that are heavy enough to settle out of solution. The alumina hydrate crystals are washed to remove remaining traces of impurities and are calcined in kilns to remove all water. The resulting alumina is ready to be reduced with the Hall–Héroult process. The alumina is melted in a cryolite bath (a molten salt of sodium–aluminum–fluoride). An electric current is passed between anodes and cathodes of carbon to

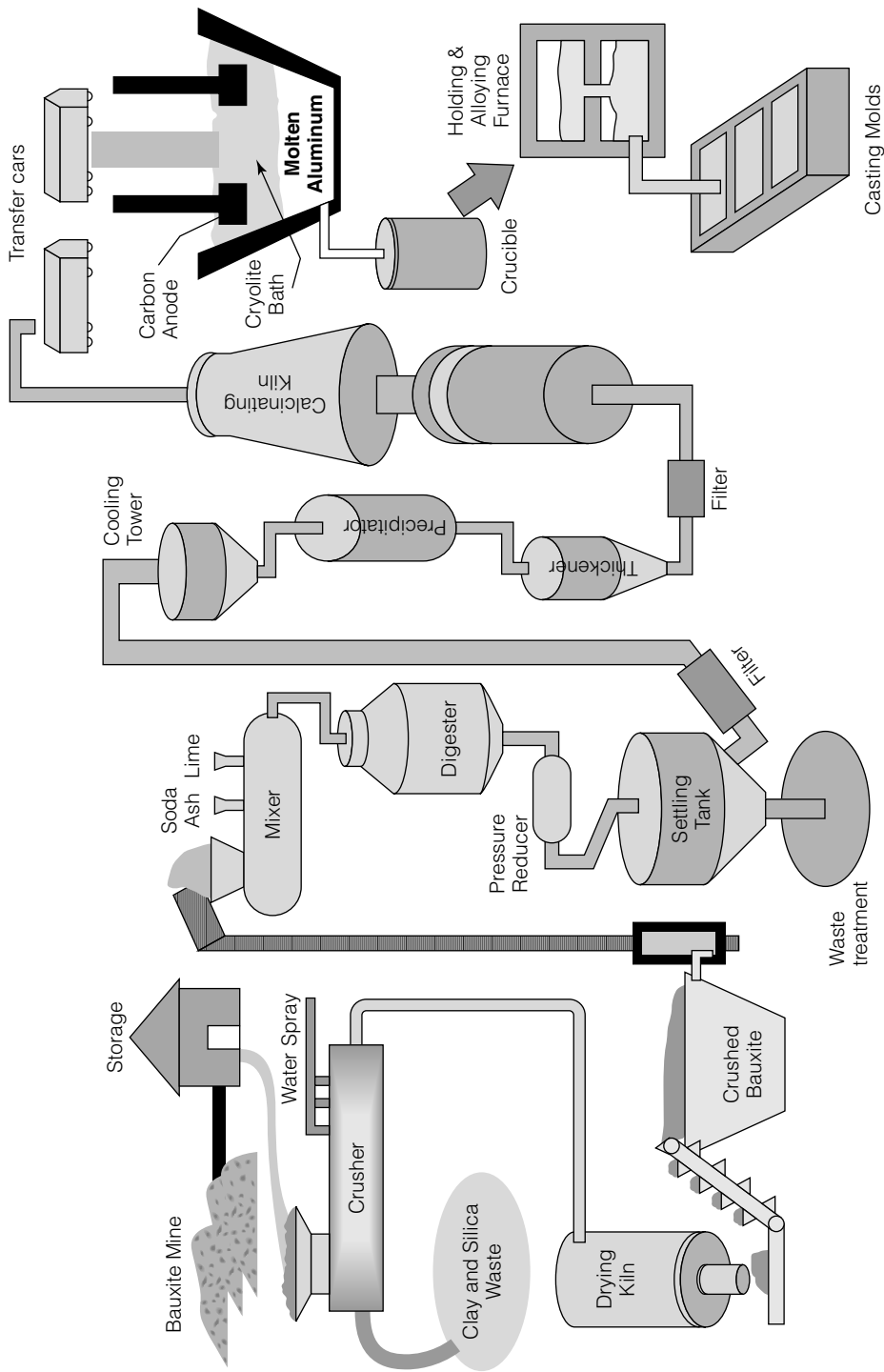


FIGURE 4.3 Aluminum production process.

separate the aluminum and oxygen molecules. The molten aluminum is collected at the cathode at the bottom of the bath. The molten aluminum, with better than 99% purity, is siphoned off to a crucible. It is then processed in a holding furnace. Hot gases are passed through the molten material to further remove any remaining impurities. Alloying elements are then added.

The molten aluminum is either shipped to a foundry for casting into finished products or is cast into ingots. The ingots are formed by a direct-chill process that produces huge sheets for rolling mills, round loglike billets for extrusion presses, or square billets for production of wire, rod, and bar stock.

Final products are made by either casting, which is the oldest process, or deforming solid aluminum stock. Three forms of casting are used: die casting, permanent mold casting, and sand casting. The basic deformation processes are forging, impact extrusion, stamping, drawing, and drawing plus ironing. Many structural shapes are made with the extrusion process. Either cast or deformed products can be machined to produce the final shape and surface texture, and they can be heat treated to alter the mechanical behavior of the aluminum. Casting and forming methods are summarized in Table 4.1.

When recycling aluminum, the scrap stock is melted in a furnace. The molten aluminum is purified and alloys are added. This process takes only about 5% of the electricity that is needed to produce aluminum from bauxite.

In addition to these conventional processes, very high strength aluminum parts can be produced using powder metallurgy methods. A powdered aluminum alloy is compacted in a mold. The material is heated to a temperature that fuses the particles into a unified solid.

4.2 Aluminum Metallurgy

Aluminum has a face center cubic (FCC) lattice structure. It is very malleable, with a typical elongation over a 50-mm (2-in.) gauge length of over 40%. It has limited tensile strength, on the order of 28 MPa (4000 psi). The modulus of elasticity of aluminum is about 69 GPa (10,000 ksi). Commercially pure aluminum (i.e., more than 99% aluminum content) is limited to nonstructural applications, such as electrical conductors, chemical equipment, and sheet metal work.

Although the strength of pure aluminum is relatively low, aluminum alloys can be as much as 15 times stronger than pure aluminum, through the addition of small amounts of alloying element, strain hardening by cold working, and heat treatment. The common alloying elements are copper, manganese, silicon, magnesium, and zinc. Cold working increases strength by causing a disruption of the slip planes in the material that resulted from the production process.

TABLE 4.1 Casting and Forming Methods for Aluminum Products (Extracted from *Reynolds Infrastructure*, 1996)

Casting Methods

Sand Casting	Sand with a binder is packed around a pattern. The pattern is removed and molten aluminum is poured in, reproducing the shape. Produces a rough texture which can be machined or otherwise surfaced if desired. Economical for low volume production and for making very large parts. Also applicable when an internal void must be formed in the product.
Permanent Mold Casting	Molten aluminum is poured into a reusable metal mold. Economical for large volume production.
Die Casting	Molten aluminum is forced into a permanent mold under high pressure. Suitable for mass production of precisely formed castings.

Forming Methods

Extrusion	Aluminum heated to 425 to 540°C (800 to 1000°F) is forced through a die. Complex cross sections are possible, including incompletely or completely enclosed voids. A variety of architectural and structural members are formed by extrusion, including tubes, pipes, I-beams, and decorative components, such as window and door frames.
Rolling	Rollers compress and elongate heated aluminum ingots, producing plates (more than 6 mm (0.25 in.) thick), sheets (0.15 to 6 mm (0.006 to 0.25 in.) thick, and foil (less than 0.15 mm (0.006 in.)).
Roll Forming	Shaping of sheet aluminum by passing stock between a series of special rollers, usually in stages. Used for mass production of architectural products, such as moldings, gutters, downspouts, roofing, siding and frames for windows and screens.
Brake Forming	Forming of sheet products with a brake press. Uses simpler tooling than roll forming but production rates are lower and the size of the product is limited.
Cutting Operations	Production of outline shapes by blanking and cutting. In blanking, a punch with the desired shape is pressed through a matching die. Used for mass production of flat shapes. Holes through a sheet are produced by piercing and perforating. Stacks of sheets can be trimmed or cut to an outline shape by a router or sheared in a guillotine-action shear.
Embossing	Shaping an aluminum sheet by pressing between mated rollers or dies, producing a raised pattern on one side and its negative indent on the other side.
Drawing	Shaping an aluminum sheet by drawing it through the gap between two mated dies in a press.
Superplastic Forming	An aluminum sheet is heated and forced over or into a mold by air pressure. Complex and deep contour shapes can be produced but the process is slow.

Figure 4.4 shows the two-phase diagram for aluminum and copper. This diagram is typical of the phase diagrams of other two-phase aluminum alloys. The alloying elements have low solubility in aluminum, and the solubility reduces as temperature drops.

As described previously, the properties of metals with this characteristic are very sensitive to heat treatments, which affect the grain size of the material and the distribution of the alloying element throughout the matrix of the lattice structures. Heat treatments typically used on aluminum alloys include annealing, hardening, aging, and stabilizing.

4.2.1 ■ Alloy Designation System

Aluminum classification starts by separating the product according to its production method, either casting or wrought methods. Aluminum alloys designed for casting are formulated to flow into the mold. Wrought aluminum alloys are used for products fabricated by deforming the aluminum into its final shape. The Aluminum Association has developed an aluminum alloy classification system shown in Table 4.2.

The designation system for wrought alloys consists of a four-digit code. The first digit indicates the alloy series. The second digit, if different from 0, indicates a modification in the basic alloy. The third and fourth digits identify the specific alloy in the series; these digits are arbitrarily assigned, except for the 1xxx series, in which the final two digits indicate the minimum aluminum content. For the 1xxx series, the aluminum content is 99% plus the

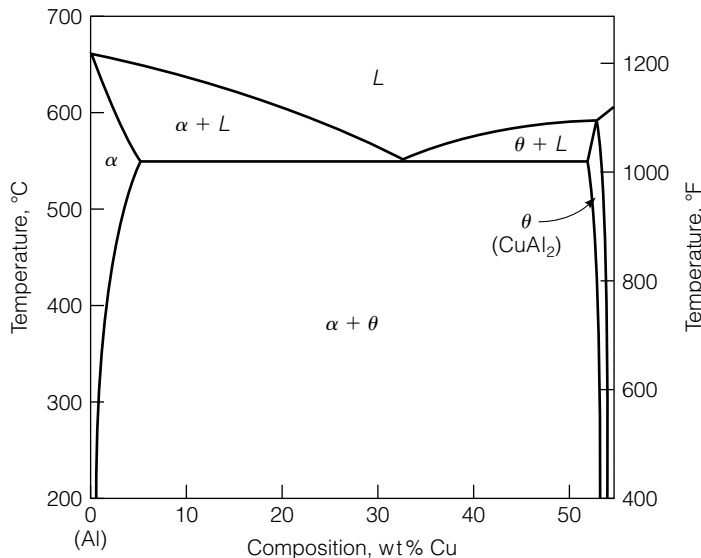


FIGURE 4.4 Aluminum-copper phase diagram.

TABLE 4.2 Designation System for Aluminum Alloys (Aluminum Association, 1993)

Wrought Aluminum Alloys		Cast Aluminum Alloys	
Alloy Series	Description or Major Alloying Elements	Alloy Series	Description or Major Alloying Elements
1xxx	99.00% Minimum Aluminum	1xx.x	99.00% Minimum Aluminum
2xxx	Copper	2xx.x	Copper
3xxx	Manganese	3xx.x	Silicon plus copper and/or magnesium
4xxx	Silicon	4xx.x	Silicon
5xxx	Magnesium	5xx.x	Magnesium
6xxx	Magnesium and silicon	6xx.x	Unused series
7xxx	Zinc	7xx.x	Zinc
8xxx	Other element	8xx.x	Tin
9xxx	Unused Series	9xx.x	Other element

last two digits of the code, expressed as a decimal fraction. For example, a 1060 contains a minimum aluminum content of 99.60%.

Cast alloys are assigned a three-digit number followed by one digit after the decimal point, as shown in Table 4.2. The first digit represents the alloy series. Note that series 3, 6, 8, and 9 have different meanings for cast versus wrought alloys. The second and third digits are arbitrarily assigned to identify specific alloys. The digit after the decimal indicates whether the alloy composition is for the final casting (xxx.0) or for ingot (xxx.1 and xxx.2).

4.2.2 ■ Temper Treatments

The mechanical properties of aluminum are greatly altered by both heat treatment and strain hardening. Therefore, specification of an aluminum material must include the manner in which the product was tempered. The processes described in Table 4.3 define the types of tempering aluminum products undergo.

Aluminum alloys used for structural applications are classified as being either heat treatable or not. Non-heat-treatable or “common” alloys contain elements that remain substantially in solid solution or that form insoluble constituents. Thus, heat treatment does not influence their mechanical properties. The properties of these alloys are dependent on the amount of cold working introduced after annealing. Heat-treatable or “strong” alloys contain elements, groups of elements, or constituents that have a considerable solid solubility at elevated temperatures and limited solubility at lower temperatures. The strength of these alloys is increased primarily by heat treatment.

4.3 Aluminum Testing and Properties

Typical properties are provided in Tables 4.4 and 4.5 for non-heat-treatable and heat-treatable wrought aluminum alloys, respectively. Typical properties for cast aluminum alloys that may be used for structural applications are given in Table 4.6. These values are only an indication of the properties of cast aluminum alloys. Material properties of cast members can vary throughout the body of the casting due to differential cooling rates.

Tests performed on aluminum are similar to those described for steel. These typically include stress-strain tensile tests to determine elastic modulus, yield strength, ultimate strength, and percent elongation. In contrast to steel, aluminum alloys do not display an upper and lower yield point. Instead, the stress-strain curve is linear up to the proportional limit, and then is a smooth curve up to the ultimate strength. Yield strength is defined based on the 0.20% strain offset method, as shown in Figure 4.5. As indicated earlier, the modulus of elasticity of aluminum alloys is on the order of 69 GPa (10,000 ksi) and is not very sensitive to types of alloys or temper treatments.

Sample Problem 4.1

An aluminum alloy rod with 10 mm diameter is subjected to a 5-kN tensile load. After the load was applied, the diameter was measured and found to be 9.997 mm. If the yield strength is 139 MPa, calculate the Poisson's ratio of the material.

Solution

$$\sigma = \frac{5000}{\pi d^2/4} = 63.7 \times 10^6 \text{ Pa} = 63.7 \text{ MPa}$$

It is clear that the applied stress is well below the yield stress and, as a result, the deformation is elastic. Hence, assume that

$$\begin{aligned} E &= 69 \text{ GPa} \\ \varepsilon_{\text{axial}} &= \frac{\sigma}{E} = \frac{63.7 \times 10^6}{69 \times 10^9} = 0.000923 \text{ m/m} \\ \Delta d &= 9.997 - 10.000 = 0.003 \text{ m} \\ \varepsilon_{\text{lateral}} &= \frac{-0.003}{10.000} = -0.0003 \text{ m/m} \\ \nu &= \frac{-\varepsilon_{\text{lateral}}}{\varepsilon_{\text{axial}}} = \frac{0.0003}{0.000923} = 0.33 \end{aligned}$$

TABLE 4.3 Temper Designations for Aluminum Alloys

Symbol	Meaning	Comment
F	as fabricated	No special control over thermal conditions or strain hardening is employed.
O	annealed	Wrought products—annealed to the lowest strength temper Cast products—annealed to improve ductility and dimensional stability. The “O” may be followed by a digit other than zero, indicating a variation with special characteristics.
H	strain hardened	<p>Wrought products only. Strength is increased by strain hardening, with or without supplemental thermal treatments. The “H” is always followed by two or more numerical digits. The first digit indicates a specific combination of basic operations. The second digit indicates the degree of strain hardening. (Codes for the second digit are 2—quarter hard, 4—half hard, 8—full hard, 9—extra hard.) When used, the third digit indicates a variation of the two digit temper. The basic operations identified by the first digit are as follows:</p> <p>H1—strain hardening only. Applies to products that are strain hardened to obtain the desired strength, without supplementary thermal treatment.</p> <p>H2—strain hardened and partial annealed. Applies to products that are strain hardened more than the desired final amount, and then reduced in strength to the desired level by partial annealing.</p> <p>H3—strain hardened and stabilized. Applies to products that are strain hardened and whose mechanical properties are stabilized either by a low temperature thermal treatment or as a result of heat introduced during fabrication. Stabilization usually improves ductility.</p>
W	solution heat treated	An unstable temper applicable only to alloys that spontaneously age at room temperature after solution heat treatment. This designation is specific only when the period of natural aging is indicated—for example, W $\frac{1}{2}$ hr,
T	thermally treated to produce stable tempers other than F, O, or H	<p>Applies to thermally treated products, with or without supplementary strain hardening to produce stable tempers. The “T” is always followed by one or two digits:</p> <p>T1—cooled from an elevated temperature shaping process and naturally aged to a substantially stable condition. Products not cold worked after cooling from an elevated temperature shaping process, or in which the effect of cold working flattening or straightening may not be recognized in mechanical property limits.</p> <p>T2—cooled from an elevated temperature shaping process, cold worked, and naturally aged to a substantially stable condition. Products cold worked to improve strength after cooling from an elevated temperature shaping process, or in which the effect of cold work in flattening or straightening is recognized in mechanical property limits.</p>

TABLE 4.3 (Continued)

Symbol	Meaning	Comment
		T3—solution heat treated, cold worked, and naturally aged to a substantially stable condition. Products cold worked to improve strength after solution heat treatment, or in which the effect of cold work in flattening or straightening is recognized in mechanical property limits.
		T4—solution heat treated and naturally aged to a substantially stable condition. Products not cold worked after solution heat treatment, or in which the effect of cold work in flattening or straightening is recognized in mechanical property limits
		T5—cooled from an elevated temperature shaping process, then artificially aged. Products not cold worked after cooling from an elevated temperature shaping process, or in which the effect of cold work in flattening or straightening may not be recognized in mechanical property limits
		T6—solution heat treated and then artificially aged. Products which are not cold worked after solution heat treatment, or in which the effect of cold work in flattening or straightening may not be recognized in mechanical property limits.
		T7—solution heat treated and overaged/stabilized. Wrought products artificially aged after solution heat treatment to carry them beyond a point of maximum strength to provide control of some significant characteristic. Cast products artificially aged after solution heat treatment to provide dimensional and strength stability.
		T8—solution heat treated, cold worked, and then artificially aged. Products cold worked to improve strength, or in which the effect of cold work in flattening or straightening is recognized in mechanical property limits.
		T9—solution heat treated, artificially aged, and then cold worked. Products cold worked to improve strength.
		T10—cooled from an elevated temperature shaping process, cold worked and then artificially aged. Products worked to improve strength, or in which the effect of cold work in flattening and straightening is recognized in mechanical property limits. Additional digits can be appended to the preceding temper designations to indicate significant variations.

TABLE 4.4 Properties of Select Non-Heat-Treatable Wrought Aluminum Alloys

Alloy	Tension				Hardness ²	Shear Ultimate		Fatigue ³ Endurance Limit		Nominal Chemical Composition	
	Ultimate		Yield			ksi	MPa	ksi	MPa		
	ksi	MPa	ksi	MPa							
1060	O	10	69	4	28	43	7	48	3	21	99.6 Al
	H-12	12	83	11	76	16	8	55	4	28	
	H-14	14	97	13	90	12	9	62	5	34	
	H-16	16	110	15	103	5	10	69	6.5	45	
	H-18	19	131	18	124	6	11	76	6.5	45	
1100	O	13	90	5	34	35	9	62	5	34	99. Al
	H-12	16	110	15	103	12	10	69	6	41	
	H-14	18	124	17	117	9	11	76	7	48	
	H-16	21	145	20	138	6	12	83	9	62	
	H-18	24	165	22	152	5	13	90	9	62	
3003	O	16	110	6	41	30	11	76	7	48	1.2 Mn
	H-12	19	131	18	124	10	12	83	8	55	
	H-14	22	152	21	145	8	14	97	9	62	
	H-16	26	179	25	172	5	15	103	10	69	
	H-18	29	200	27	186	4	16	110	10	69	

TABLE 4.4 (Continued)

5005	O	18	124	6	41	25	28	11	76		0.8 Mg
	H-12	20	138	19	131	10		14	97		
	H-14	23	159	22	152	6		14	97		
	H-16	26	179	25	172	5		15	103		
	H-18	29	200	28	193	4		16	110		
	H-32	20	138	17	117	11	36	14	97		
	H-34	23	159	20	138	8	41	14	97		
	H-36	26	179	24	165	6	46	15	103		
5086	H-38	29	200	27	186	5	51	16	110		
	O	38	262	17	117	22	30	23	159	21	145 Mg
	H-32	42	290	30	207	12	72	25	172	22	152 0.45 Mn
	H-34	47	324	37	255	10	82	27	186	23	159
	H-111	40	276	27	186		65	23	159	21	145
	H-112	39	269	19	131	14	64	23	159	21	145
	H-116	42	290	30	207		72	25	173	22	152
	O	45	310	23	159	20	70	27	186	22	152 5.1Mg
5456	H-111	47	324	33	228		75	27	186	24	165 0.7Mn
	H-112	45	310	24	156		70	27	186		0.12 Cr
	H-116	51	352	37	255		90	30	207	23	159

¹ percent elongation over 2 in.

² Brinell number, 500-kg load

³ 500,000,000 cycles of complete stress reversal using R.R. Moore type of machine and specimen.

TABLE 4.5 Properties of Select Heat-Treatable Wrought Aluminum Alloys

Alloy	Tension				Hardness ²	Shear Ultimate		Fatigue ³ Endurance Limit		Nominal Chemical Composition		
	Ultimate		Yield			ksi	MPa	ksi	MPa			
	ksi	MPa	ksi	MPa							1/16"	1/2"
2014	O	27	186	14	97	18	45	18	124	13	90	4.5 Cu, 0.8 Mn
	T4/T451	62	427	42	290	50	105	38	262	20	138	0.8 Si, 0.4 M
	T6/T651	70	483	60	414	13	135	42	290	18	124	
6053	O	16	110	8	55	35	26	11	76	8	55	1.2 Mg,
	T6	37	255	32	221	13	80	23	159	12	90	0.25 CR
6061	O	18	124	8	55	30	30	12	83	9	62	1.0 Mg, 0.6 Si
	T4/T451	35	241	21	145	22	65	24	165	14	97	0.25 Cu,
	T6/T651	45	310	40	276	12	95	30	207	14	97	0.25 Cr
6063	O	13	90	7	48		25	10	69	8	55	0.7 Mg
	T1	22	152	13	90	20	42	14	97	9	62	0.4 Si
	T4	25	172	13	90	22		16	110			
	T5	27	186	21	145	12	60	17	117	10	69	
	T6	35	241	31	214	12	73	22	152	10	69	
7178	T83	37	255	35	241	9	82	22	152			
	T831	30	207	27	186	10	70	18	124			
	T832	42	290	39	269	12	95	27	186			
	O	33	228	15	103	15	60	22	152			6.8 Zn, 2.0 Cu
	T6/T651	88	607	78	538	10	160	52	359	22	152	2.7 Mg, 0.3 Mn
T76/ T765	83	572	73	503	11							

¹ Percent elongation over 2 in.

² Brinell number, 500-kg load

³ 500,000,000 cycles of complete stress reversal using R.R. Moore type of machine and specimen.

TABLE 4.6 Typical Properties of Select Cast Aluminum Alloys

Cast Alloy Designation	Tension					Hardness ²	Shear Ultimate		Fatigue ³ Endurance Limit	
	Ultimate		Yield		Elongation ¹		ksi	MPa	ksi	MPa
	ksi	MPa	ksi	MPa						
356.0-T6 ⁴	40	276	27	186	5	90	32	221	13	90
356.0-T7 ⁴	33	228	24	165	5	70	25	172	11	76
A356.0-T61 ⁴	41	283	30	207	10	80				
A357.0-T6 ⁴	50	345	40	276	10	85	43	296	16	110
A444.0-T4 ⁴	23	159	10	69	21	45				
356.0-T6 ⁵	33	228	24	165	3.5	70	26	179	8.5	59
356.0-T7 ⁵	34	234	30	207	2.0	75	24	165	9.0	62
Almag 35 535.0 ⁵	40	276	21	145	13	70	28	193	10	69

¹Percent elongation over 2 in.

²Brinell number, 500-kg load

³500,000,000 cycles of complete stress reversal using R.R. Moore type of machine and specimen.

⁴Permanent mold

⁵Sand casting

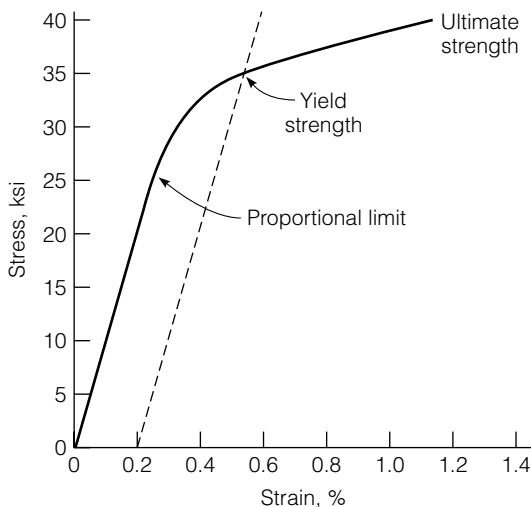


FIGURE 4.5 Aluminum stress-strain diagram

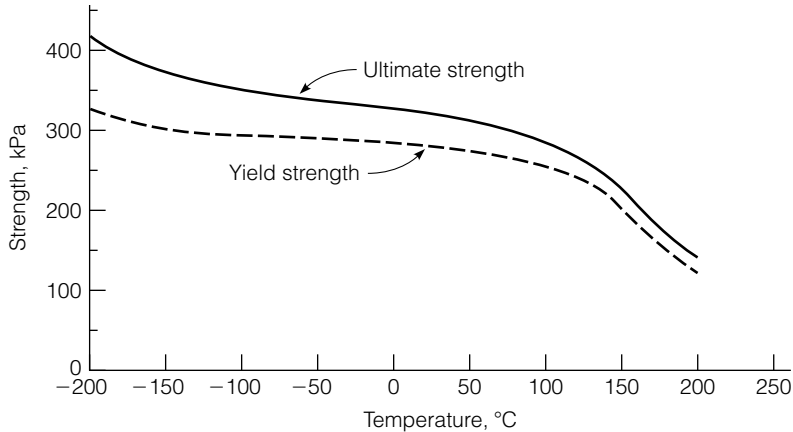


FIGURE 4.6 Tensile strength of aluminum at different temperatures. (Courtesy of the Aluminum Association, 1987.)

Aluminum's coefficient of thermal expansion is $0.000023/^{\circ}\text{C}$ ($0.000013/^{\circ}\text{F}$), about twice as large as that of steel and concrete. Thus, joints between aluminum and steel or concrete must be designed to accommodate the differential movement.

Strengths of aluminum are considerably affected by temperature, as shown in Figure 4.6. At temperatures above 150°C (300°F), tensile strengths are reduced considerably. The temperature at which the reduction begins and the extent of the reduction depends on the alloy. At temperatures below room temperature, aluminum becomes stronger and tougher as the temperature decreases.

4.4 Welding and Fastening

Aluminum pieces can be joined either by welding or by using fasteners. Welding requires that the tough oxide coating on aluminum be broken and kept from reforming during welding, so arc welding is generally performed in the presence of an inert gas that shields the weld from oxygen in the atmosphere. The two common processes by which aluminum is welded are gas metal arc welding, GMAW, and gas tungsten arc welding, GTAW. In the GMAW process, the filler wire also serves as the electrode. GTAW uses a tungsten electrode and a separate filler wire. Welding can alter the tempering of the aluminum in the area of the weld. For example, the tensile strength of 6061-T6 is 290 MPa (42 ksi), but the tensile strength of a weld in this alloy is only about 165 MPa (24 ksi). For design purposes, it is assumed the weld affects an area of 25 mm (1 in.) on each side of the weld.

In addition to welding, either bolts or rivets can join aluminum pieces. Bolts can be either aluminum or steel. When steel bolts are used, they must

be either galvanized, aluminized, cadmium plated, or made of stainless steel to prevent the development of galvanic corrosion. Rivet fasteners are made of aluminum and are cold driven. Both bolt and rivet joints are designed based on the shear strength of the fastener and the bearing strength of the material being fastened.

4.5 Corrosion

Aluminum develops a thin oxidation layer immediately upon exposure to the atmosphere. This tough oxide film protects the surface from further oxidation. The alloying elements alter the corrosion resistance of the aluminum. The alloys used for airplanes are usually given extra protection by painting or “cladding” with a thin coat of a corrosion-resistant alloy. Painting is generally not needed for medium-strength alloys used for structural applications.

Galvanic corrosion occurs when aluminum is in contact with any of several metals in the presence of an electrical conductor, such as water. The best protection for this problem is to break the path of the galvanic cell by painting, using an insulator, or keeping the dissimilar metals dry.

S U M M A R Y

Although aluminum has many desirable attributes, its use as a structural material in civil engineering has been limited, primarily by economic considerations and a lack of performance information. Aluminum alloys and heat treatments provide products with a wide range of characteristics. The advantages of aluminum relative to steel include lightweight, high strength-to-weight ratio, and corrosion resistance.

Q U E S T I O N S A N D P R O B L E M S

- 4.1 Name the two primary factors that make aluminum an attractive structural engineering material.
- 4.2 Compare the strength and modulus of elasticity of aluminum alloys with those of steel.

- 4.3 An aluminum alloy specimen with a radius of 0.28 in. was subjected to tension until fracture and produced results shown in Table P4.3.

Table P4.3

Stress, ksi	Strain, 10 ⁻³ in./in.
8	0.6
17	1.5
27	2.4
35	3.2
43	4.0
50	4.6
58	5.2
62	5.8
64	6.2
65	6.5
67	7.3
68	8.1
70	9.7

- Using a spreadsheet program, plot the stress–strain relationship.
 - Calculate the modulus of elasticity of the aluminum alloy.
 - Determine the proportional limit.
 - What is the maximum load if the stress in the bar is not to exceed the proportional limit?
 - Determine the 0.2% offset yield strength.
 - Determine the tensile strength.
 - Determine the percent of elongation at failure.
- 4.4 A round aluminum alloy bar with a 0.5-inch diameter and 2-inch gauge length was subjected to tension to fracture. The load and deformation data were as shown in Table P4.4.

Table P4.4

Load (lb)	ΔL (in.)	Load (lb)	ΔL (in.)
0	0.0000	10,853	0.0136
1395	0.0014	11,461	0.0168
2800	0.0028	12,050	0.0220
4195	0.0042	12,599	0.0310
5600	0.0055	12,953	0.0420
7010	0.0070	13,188	0.0528
8282	0.0083	13,345	fracture
9852	0.0103		

Using a spreadsheet program, obtain the following:

- A plot of the stress–strain relationship. Label the axes and show units.

- b. A plot of the linear portion of the stress–strain relationship. Determine modulus of elasticity using the best fit approach.
- c. Proportional limit.
- d. Yield stress at an offset strain of 0.002 in/in.
- e. Tangent modulus at a stress of 60 ksi.
- f. Secant modulus at a stress of 60 ksi.
- 4.5 An aluminum alloy bar with a rectangular cross section that has a width of 12.5 mm, thickness of 6.25 mm, and a gage length of 50 mm was tested in tension to fracture according to ASTM E-8 method. The load and deformation data were as shown in Table P4.5.

Table P4.5

Load (kN)	ΔL (mm)	Load (kN)	ΔL (mm)
0	0	33.5	1.486
3.3	0.025	35.3	2.189
14.0	0.115	37.8	3.390
25.0	0.220	39.8	4.829
29.0	0.406	40.8	5.961
30.6	0.705	41.6	7.386
31.7	0.981	41.2	8.047
32.7	1.245		

Using a spreadsheet program, obtain the following:

- a. A plot of the stress–strain relationship. Label the axes and show units.
- b. A plot of the linear portion of the stress–strain relationship. Determine the modulus of elasticity using the best fit approach.
- c. Proportional limit.
- d. Yield stress at an offset strain of 0.002 in/in.
- e. Tangent modulus at a stress of 450 MPa.
- f. Secant modulus at a stress of 450 MPa.
- 4.6 A round aluminum alloy bar with a 0.25-inch diameter and a 1-inch gauge length was tested in tension to fracture according to ASTM E-8 method. The load and deformation data were as shown in Table P4.6.

Table P4.6

Load (lb)	Displacement (in.)	Load (lb)	Displacement (in.)
0	0	2957	0.02926
288	0.00050	3119	0.04310
1239	0.00225	3337	0.06674
2207	0.00432	3513	0.09506
2562	0.00799	3604	0.11734
2703	0.01388	3677	0.14539
2800	0.01930	3643	0.15841
2886	0.02451		

Using a spreadsheet program, obtain the following:

- a. A plot of the stress–strain relationship. Label the axes and show units.
 - b. A plot of the linear portion of the stress–strain relationship. Determine modulus of elasticity using the best fit approach.
 - c. Proportional limit.
 - d. Yield stress at an offset strain of 0.002 in/in.
 - e. Initial tangent modulus:
 1. If the specimen is loaded to 3200 lb only and then unloaded, what is the permanent change in gage length?
 2. When the applied load was 1239 lb, the diameter was measured as 0.249814 inches. Determine Poisson’s ratio.
- 4.7 An aluminum alloy rod has a circular cross section with a diameter of 8 millimeters. This rod is subjected to a tensile load of 4 kN. Assume $E = 69$ GPa.
- a. What will be the lateral strain if Poisson’s ratio is 0.33?
 - b. What will be the diameter after load application?
- 4.8 A 3003-H14 aluminum alloy rod with 0.5 in. diameter is subjected to 2000-lb tensile load. Calculate the resulting diameter of the rod. If the rod is subjected to a compressive load of 2000 lb, what will be the diameter of the rod? Assume that the modulus of elasticity is 10,000 ksi, Poisson’s ratio is 0.33, and the yield strength is 21 ksi.
- 4.9 The stress–strain relation of an aluminum alloy bar having a length of 2 m and a diameter of 10 mm is expressed by the equation

$$\varepsilon = \frac{\sigma}{70,000} \left[1 + \frac{3}{7} \left(\frac{\sigma}{270} \right)^9 \right]$$

where σ is in MPa. If the rod is axially loaded by a tensile force of 20 kN and then unloaded, what is the permanent deformation of the bar?

- 4.10 A tension test was performed on an aluminum alloy specimen to fracture. The original diameter of the specimen is 0.5 in. and the gage length is 2.0 in. The information obtained from this experiment consists of applied tensile load (P) and increase in length (ΔL). The results are tabulated in Table P4.10. Using a spreadsheet program, complete the table by calculating engineering stress (σ) and engineering strain (ε). Determine the toughness of the material (u_t) by calculating the area under the stress–strain curve, namely,

$$u_t = \int_0^{\varepsilon_f} \sigma \, d\varepsilon$$

where ε_f is the strain at fracture. The preceding integral can be approximated numerically using a trapezoidal integration technique:

$$u_t = \sum_{i=1}^n u_i = \sum_{i=1}^n \frac{1}{2} (\sigma_i + \sigma_{i-1}) (\varepsilon_i - \varepsilon_{i-1})$$

Table P4.10

Observation No.	P (lb)	ΔL (in.)	σ (psi)	ε (in./in.)	u_i (psi)
0	0	0			N/A
1	1181	0.0015			
2	2369	0.003			
3	3550	0.0045			
4	4738	0.0059			
5	5932	0.0075			
6	7008	0.0089			
7	8336	0.011			
8	9183	0.0146			
9	9698	0.018			
10	10,196	0.0235			
11	10,661	0.0332			
12	10,960	0.0449			
13	11,159	0.0565			
14	11,292	0.0679			

$$u_i =$$

4.11 Discuss galvanic corrosion of aluminum. How can aluminum be protected from galvanic corrosion?

4.6 References

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5

AGGREGATES



There are two main uses of aggregates in civil engineering: as an underlying material for foundations and pavements and as ingredients in portland cement and asphalt concretes. By dictionary definition, aggregates are a combination of distinct parts gathered into a mass or a whole. Generally, in civil engineering the term *aggregate* means a mass of crushed stone, gravel, sand, etc., predominantly composed of individual particles, but in some cases including clays and silts. The largest particle size in aggregates may have a diameter as large as 150 mm (6 in.) and the smallest particle can be as fine as 5 to 10 microns. The balance of this chapter presents information about aggregates as used in construction. Information is not presented about the characteristics and properties of soils, as this is the purview of textbooks on geotechnical engineering.

5.1 Aggregate Sources

Natural sources for aggregates include gravel pits, river run deposits, and rock quarries. Generally, *gravel* comes from pits and river deposits, whereas *crushed stones* are the result of processing rocks from quarries. Usually, gravel deposits must also be crushed to obtain the needed size distribution, shape, and texture (Figure 5.1).

Manufactured aggregates can use slag waste from steel mills and expanded shale and clays to produce lightweight aggregates. Heavyweight concrete, used for radiation shields, can use steel slugs and bearings for the aggregate. Styrofoam beads can be used as an aggregate in lightweight concrete used for insulation.



FIGURE 5.1 Aggregate stockpiling.

5.2 Geological Classification

All natural aggregates result from the breakdown of large rock masses. Geologists classify rocks into three basic types: *igneous*, *sedimentary*, and *metamorphic*. Volcanic action produces igneous rocks by hardening or crystallizing molten material, magma. The magma cools either at the earth's surface, when it is exposed to air or water, or within the crust of the earth. Cooling at the surface produces *extrusive* igneous rocks, while cooling underground produces *intrusive* igneous rocks. In general, the extrusive rocks cool much more rapidly than the intrusive rocks. Therefore, we would expect extrusive igneous rocks to have a fine grain size and potentially to include air voids and other inclusions. Intrusive igneous rocks have larger grain sizes and fewer flaws. Igneous rocks are classified based on grain size and composition. Coarse grains are larger than 2 mm and fine grains are less than 0.2 mm. Classification based on composition is a function of the silica content, specific gravity, color, and the presence of free quartz.

Sedimentary rocks coalesce from deposits of disintegrated existing rocks or inorganic remains of marine animals. Wind, water, glaciers, or direct chemical precipitation transport and deposit layers of material that become sedimentary rocks, resulting in a stratified structure. Natural cementing binds the particles together. Classification is based on the predominant mineral

present: calcareous (limestone, chalk, etc.), siliceous (chert, sandstone, etc.), and argillaceous (shale, etc.).

Metamorphic rocks form from igneous or sedimentary rocks that are drawn back into the earth's crust and exposed to heat and pressure, re-forming the grain structure. Metamorphic rocks generally have a crystalline structure, with grain sizes ranging from fine to coarse.

All three classes of rock are used successfully in civil engineering applications. The suitability of aggregates from a given source must be evaluated by a combination of tests to check physical, chemical, and mechanical properties, and must be supplemented by mineralogical examination. The best possible prediction of aggregate suitability for a given application is that based on historical performance in a similar design.

5.3 Evaluation of Aggregate Sources

Civil engineers select aggregates for their ability to meet specific project requirements, rather than their geologic history. The physical and chemical properties of the rocks determine the acceptability of an aggregate source for a construction project. These characteristics vary within a quarry or gravel pit, making it necessary to continually sample and test the materials as the aggregates are being produced.

Due to the quantity of aggregates required for a typical civil engineering application, the cost and availability of the aggregates are important when selecting an aggregate source. Frequently, one of the primary challenges facing the materials engineer on a project is how to use the locally available material in the most cost-effective manner.

Potential aggregate sources are usually evaluated for quality of the larger pieces, the nature and amount of fine material, and the gradation of the aggregate. The extent and quality of rock in the quarry is usually investigated by drilling cores and performing trial blasts (or shots) to evaluate how the rock breaks and by crushing some materials in the laboratory to evaluate grading, particle shape, soundness, durability, and amount of fine material. Cores are examined petrographically for general quality, suitability for various uses, and amount of deleterious materials. Potential sand and gravel pits are evaluated by collecting samples and performing sieve analysis tests. The amount of large gravel and cobble sizes determines the need for crushing, while the amount of fine material determines the need for washing. Petrographic examinations evaluate the nature of aggregate particles and the amount of deleterious material (Meininger and Nichols 1990).

Price and availability are universal criteria that apply to all uses of aggregates. However, the required aggregate characteristics depend on how they will be used in the structure; they may be used as base material, in asphalt concrete, or in portland cement concrete.

5.4 Aggregate Uses

As mentioned, aggregates are primarily used as an underlying material for foundations and pavements and as ingredients in portland cement and asphalt concretes. Aggregate underlying materials, or base courses, can add stability to a structure, provide a drainage layer, and protect the structure from frost damage (Figure 5.2). Stability is a function of the interparticle friction between the aggregates and the amount of clay and silt “binder” material in the voids between the aggregate particles. However, increasing the clay and silt content will block the drainage paths between the aggregate particles, thereby inhibiting the ability of the material to act as a drainage layer.

In portland cement concrete, 60% to 75% of the volume and 79% to 85% of the weight is made up of aggregates. The aggregates act as a filler to reduce the amount of cement paste needed in the mix. In addition, aggregates have greater volume stability than the cement paste. Therefore, maximizing the amount of aggregate, to a certain extent, improves the quality and economy of the mix.

In asphalt concrete, aggregates constitute over 80% of the volume and 92% to 96% of the mass. The asphalt cement acts as a binder to hold the aggregates together, but does not have enough strength to lock the aggregate particles into position. As a result, the strength and stability of asphalt



FIGURE 5.2 Compacted aggregate base before placing the hot-mix asphalt or portland cement concrete layer of a paved road.

concrete depends mostly on interparticle friction between the aggregates and, to a limited extent, on the binder.

5.5 Aggregate Properties

Aggregates' properties are defined by the characteristics of both the individual particles and the characteristics of the combined material. These properties can be further described by their physical, chemical, and mechanical characteristics, as shown in Table 5.1 (Meininger and Nichols, 1990). There are several individual particle characteristics that are important in determining if an aggregate source is suitable for a particular application. Other characteristics are measured for designing portland cement and asphalt concrete mixes (Goetz and Wood 1960).

5.5.1 Particle Shape and Surface Texture

The shape of the individual aggregate particles, Figures 5.3 and 5.4, determines how the material will pack into a dense configuration and also determines the mobility of the stones within a mix. There are two considerations in the shape of the material: *angularity* and *flakiness*. Crushing rocks produces angular particles with sharp corners. Due to weathering, the corners of the aggregates break down, creating *subangular* particles. When the aggregates tumble while being transported in water, the corners can become completely *rounded*. Generally, angular aggregates produce bulk materials with higher stability than rounded aggregates. However, the angular aggregates will be more difficult to work into place than rounded aggregates, since their shapes make it difficult for them to slide across each other. Flakiness describes the relationship between the smallest and largest dimensions of the aggregate.

The roughness of the aggregate surface plays an important role in the way the aggregate compacts and bonds with the binder material. Aggregates with a *rough* texture are more difficult to compact into a dense configuration than *smooth* aggregates. Rough texture generally improves bonding and increases interparticle friction. In general, natural gravel and sand have a smooth texture, whereas crushed aggregates have a rough texture.

For the purpose of preparing portland cement concrete, it is desirable to use rounded and smooth aggregate particles to improve the workability of fresh concrete during mixing. However, angular and rough particles are desirable for asphalt concrete and base courses in order to increase the stability of the materials in the field and to reduce rutting. Flaky and elongated aggregates are undesirable for asphalt concrete, since they are difficult to compact during construction and are easy to break.

Many specifications for aggregates used in asphalt concrete require a minimum percentage of aggregates with crushed faces as a surrogate *shape*

TABLE 5.1 Basic Aggregate Properties (Meininger and Nichols, 1990)

Property	Relative Importance for End Use*		
	Portland Cement Concrete	Asphalt Concrete	Base
PHYSICAL			
Particle shape (angularity)	M	V	V
Particle shape (flakiness, elongation)	M	M	M
Particle size—maximum	M	M	M
Particle size—distribution	M	M	M
Particle surface texture	M	V	V
Pore structure, porosity	V	M	U
Specific gravity, absorption	V	M	M
Soundness—weatherability	V	M	M
Unit weight, voids—loose, compacted	V	M	M
Volumetric stability—thermal	M	U	U
Volumetric stability—wet/dry	M	U	M
Volumetric stability—freeze/thaw	V	M	M
Integrity during heating	U	M	U
Deleterious constituents	V	M	M
CHEMICAL			
Solubility	M	U	U
Surface charge	U	V	U
Asphalt affinity	U	V	M
Reactivity to chemicals	V	U	U
Volume stability—chemical	V	M	M
Coatings	M	M	U
MECHANICAL			
Compressive strength	M	U	U
Toughness (impact resistance)	M	M	U
Abrasion resistance	M	M	M
Character of products of abrasion	M	M	U
Mass stability (stiffness, resilience)	U	V	V
Polishability	M	M	U

*V = Very important M = Moderately important U = Unimportant or importance unknown

and *texture requirement*. A crushed particle exhibits one or more mechanically induced fractured faces and typically has a rough surface texture. To evaluate the angularity and surface texture of coarse aggregate, the percentages of particles with one and with two or more crushed faces are counted in a representative sample.

For fine aggregate, angularity and surface texture can be measured indirectly using the ASTM C1252 method, Test Method for Uncompacted Void Content of Fine Aggregate. In this test a sample of fine aggregate is poured into a small cylinder by flowing it through a standard funnel, as shown in

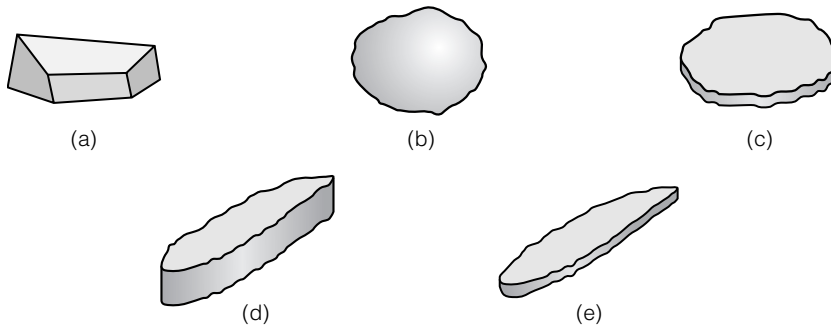


FIGURE 5.3 Particle shapes: (a) angular, (b) rounded, (c) flaky, (d) elongated, and (e) flaky and elongated.



FIGURE 5.4 Angular and rounded aggregates.

Figure 5.5. By determining the weight of the fine aggregate in the filled cylinder of known volume, the void content can be calculated as the difference between the cylinder volume and the fine aggregate volume collected in the cylinder. The volume of the fine aggregate is calculated by dividing the weight of the fine aggregate by its bulk density. The higher the amount of void content, the more angular and the rougher will be the surface texture of the fine aggregate.

5.5.2 ■ Soundness and Durability

The ability of aggregate to withstand weathering is defined as soundness or durability. Aggregates used in various civil engineering applications must be sound and durable, particularly if the structure is subjected to severe climatic conditions. Water freezing in the voids of aggregates generates stresses that can fracture the stones. The soundness test (ASTM C88) simulates weathering by soaking the aggregates in either a sodium sulfate or a magnesium

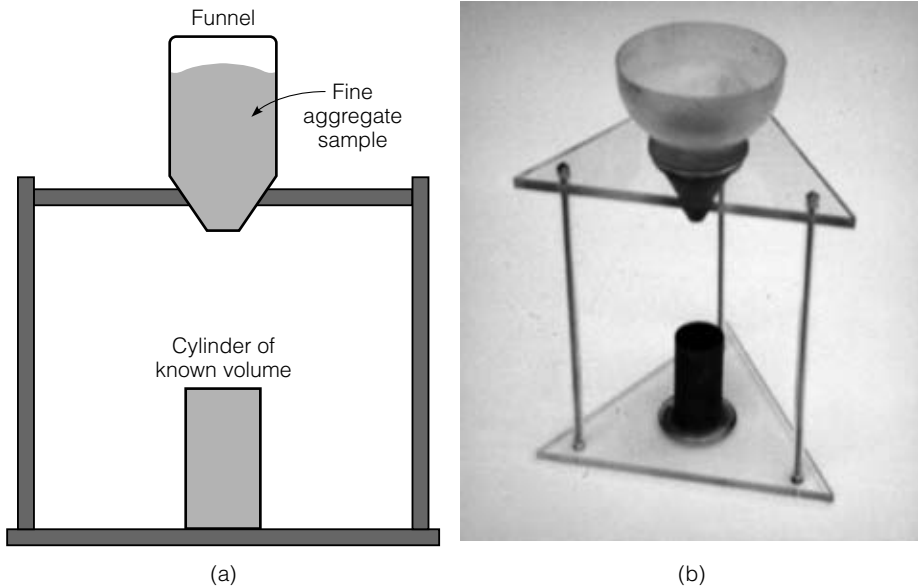


FIGURE 5.5 Apparatus used to measure angularity and surface texture of fine aggregate.

sulfate solution. These sulfates cause crystals to grow in the aggregates, simulating the effect of freezing. The test starts with an oven-dry sample separated into different sized fractions. The sample is subjected to cycles of soaking in the sulfate for 16 hours followed by drying. Typically, the samples are subjected to five cycles. Afterwards, the aggregates are washed and dried, each size is weighed, and the weighted average percentage loss for the entire sample is computed. This result is compared with allowable limits to determine whether the aggregate is acceptable. This is an empirical screening procedure for new aggregate sources when no service records are available.

The soundness by freeze thaw (AASHTO T103) and potential expansion from hydrated reactions (ASTM D4792) are alternative screening tests for evaluating soundness. The durability of aggregates in portland cement concrete can be tested by rapid freezing and thawing (ASTM C666), critical dilation by freezing (ASTM C671), and by frost resistance of coarse aggregates in air-entrained concrete by critical dilation (ASTM C682).

5.5.3 ■ Toughness, Hardness, and Abrasion Resistance

The ability of aggregates to resist the damaging effect of loads is related to the hardness of the aggregate particles and is described as the toughness or abrasion resistance. The aggregate must resist crushing, degradation, and disintegration when stockpiled, mixed as either portland cement or asphalt concrete, placed and compacted, and exposed to loads.



FIGURE 5.6 Los Angeles abrasion machine.

The Los Angeles abrasion test (ASTM C131, C535) evaluates the aggregates' toughness and abrasion resistance. In this test, aggregates blended to a fixed size distribution are placed in a large steel drum with standard sized steel balls that act as an abrasive charge (see Figure 5.6). The drum is rotated, typically for 500 revolutions. The material is recovered from the machine and passed through a sieve that retains all of the original material. The percentage weight loss is the LA abrasion number. This is an empirical test; that is, the test results do not have a scientific basis and are meaningful only when local experience defines the acceptance criteria.

5.5.4 ■ Absorption

Although aggregates are inert, they can capture water and asphalt binder in surface voids. The amount of water the aggregates absorb is important in the design of portland cement concrete, since moisture captured in the aggregate voids is not available to improve the workability of the plastic concrete and to react with the cement. There is no specific level of aggregate absorption that is desirable for aggregates used in portland cement concrete, but aggregate absorption must be evaluated to determine the appropriate amount of water to mix into the concrete.

Absorption is also important for asphalt concrete, since absorbed asphalt is not available to act as a binder. Thus, highly absorptive aggregates require greater amounts of asphalt binder, making the mix less economical.

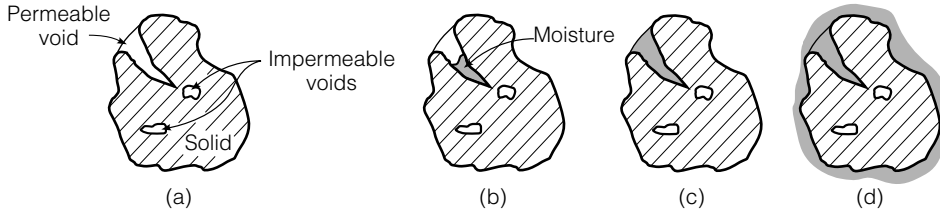


FIGURE 5.7 Voids and moisture absorption of aggregates: (a) bone dry, (b) air dry, (c) saturated surface-dry (SSD), and (d) moist.

On the other hand, some asphalt absorption is desired to promote bonding between the asphalt and the aggregate. Therefore, low-absorption aggregates are desirable for asphalt concrete.

Figure 5.7 demonstrates the four moisture condition states for an aggregate particle. *Bone dry* means the aggregate contains no moisture; this requires drying the aggregate in an oven to a constant mass. In an *air dry* condition, the aggregate may have some moisture but the saturation state is not quantified. In a *saturated surface-dry (SSD)* condition, the aggregate's voids are filled with moisture but the main surface area of the aggregate particles is dry. *Absorption* is defined as the moisture content in the SSD condition. Moist aggregates have a moisture content in excess of the SSD condition. Free moisture is the difference between the actual moisture content of the aggregate and the moisture content in the SSD condition.

Sample Problem 5.1

A sample of sand has the following properties:

$$\begin{aligned}\text{Wet mass} &= 625.2 \text{ g} \\ \text{Dry mass} &= 589.9 \text{ g} \\ \text{Absorption} &= 1.6\%\end{aligned}$$

Determine: (a) total moisture content, and (b) free moisture content

Solution

$$\text{a. Mass of water} = 625.2 - 589.9 = 35.3 \text{ g}$$

$$\text{Total moisture content} = \frac{35.3}{589.9} \times 100 = 6.0\%$$

$$\text{b. Free moisture} = 6.0 - 1.6 = 4.4\%$$

5.5.5 ■ Specific Gravity

The weight–volume characteristics of aggregates are not an important indicator of aggregate quality, but they are important for concrete mix design. *Density*, the mass per unit volume, could be used for these calculations. However, *specific gravity* (Sp. Gr.), the mass of a material divided by the mass of an equal volume of distilled water, is more commonly used. Four types of specific gravity are defined based on how voids in the aggregate particles are considered. Three of these types—*bulk-dry*, *bulk-saturated surface-dry*, and *apparent* specific gravity—are widely accepted and used in portland cement and asphalt concrete mix design. These are defined as

$$\text{Bulk Dry Sp. Gr.} = \frac{\text{Dry Weight}}{(\text{Total Particle Volume})\gamma_w} = \frac{W_s}{(V_s + V_i + V_p)\gamma_w} \quad (5.1)$$

$$\text{Bulk SSD Sp. Gr.} = \frac{\text{SSD Weight}}{(\text{Total Particle Volume})\gamma_w} = \frac{W_s + W_p}{(V_s + V_i + V_p)\gamma_w} \quad (5.2)$$

$$\text{Apparent Sp. Gr.} = \frac{\text{Dry Weight}}{(\text{Volume Not Accessible to Water})\gamma_w} = \frac{W_s}{(V_s + V_i)\gamma_w} \quad (5.3)$$

where

W_s = weight of solids

V_s = volume of solids

V_i = volume of water impermeable voids

V_p = volume of water permeable voids

W_p = weight of water in the permeable voids when the aggregate is in the SSD condition

γ_w = unit weight of water

Figure 5.8 shows that, when aggregates are mixed with asphalt binder, only a portion of the water-permeable voids are filled with asphalt. Hence, a fourth type of specific gravity—the *effective specific gravity*—is defined as

$$\text{Effective Sp.Gr.} = \frac{\text{Dry weight}}{(\text{Volume not accessible to asphalt})\gamma_w} = \frac{W_s}{(V_s + V_c)\gamma_w} \quad (5.4)$$

where V_c is volume of voids not filled with asphalt cement.

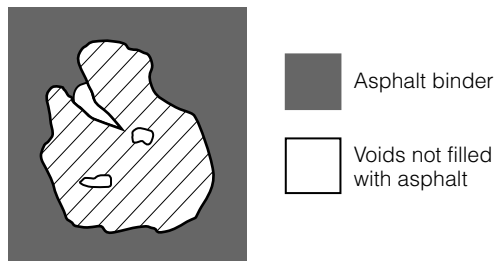


FIGURE 5.8 Aggregate particle submerged in asphalt cement; not all voids are filled with asphalt.

At present, there is no standard method for determining the effective specific gravity of aggregates directly. The U.S. Corps of Engineers has defined a method for determining the effective specific gravity of aggregates that absorb more than 2.5% water.

The specific gravity and absorption of coarse aggregates are determined in accordance with ASTM C127. In this procedure, a representative sample of the aggregate is soaked for 24 hours and weighed suspended in water. The sample is then dried to the SSD condition and weighed. Finally, the sample is dried to a constant weight and weighed. The specific gravity and absorption are determined by

$$\text{Bulk Dry Sp. Gr.} = \frac{A}{B - C} \quad (5.5)$$

$$\text{Bulk SSD Sp. Gr.} = \frac{B}{B - C} \quad (5.6)$$

$$\text{Apparent Sp. Gr.} = \frac{A}{A - C} \quad (5.7)$$

$$\text{Absorption (\%)} = \frac{B - A}{A}(100) \quad (5.8)$$

where

A = dry weight

B = SSD weight

C = submerged weight

ASTM C128 defines the procedure for determining the specific gravity and absorption of fine aggregates. A representative sample is soaked in water for 24 hours and dried back to the SSD condition. A 500-g sample of the SSD material is placed in a *pycnometer*, a constant volume flask; water is added to the constant volume mark on the pycnometer and the weight is determined again. The sample is then dried and the weight is determined. The specific gravity and absorption are determined by

$$\text{Bulk Dry Sp. Gr.} = \frac{A}{B + S - C} \quad (5.9)$$

$$\text{Bulk SSD Sp. Gr.} = \frac{S}{B + S - C} \quad (5.10)$$

$$\text{Apparent Sp. Gr.} = \frac{A}{B + A - C} \quad (5.11)$$

$$\text{Absorption (\%)} = \frac{S - A}{A}(100) \quad (5.12)$$

where

A = dry weight

B = weight of the pycnometer filled with water

C = weight of the pycnometer filled with aggregate and water

S = saturated surface—dry weight of the sample

5.5.6 Bulk Unit Weight and Voids in Aggregate

The bulk unit weight of aggregate is needed for the proportioning of portland cement concrete mixtures. According to ASTM C29 procedure, a rigid container of known volume is filled with aggregate, which is compacted either by rodding, jiggling, or shoveling. The bulk unit weight of aggregate (γ_b) is determined as

$$\gamma_b = \frac{W_s}{V} \quad (5.13)$$

where W_s is the weight of aggregate (stone) and V is the volume of the container.

If the bulk dry specific gravity of the aggregate (G_{sb}) (ASTM C127 or C128) is known, the percentage of voids between aggregate particles can be determined as follows:

$$\begin{aligned} \%V_s &= \frac{V_s}{V} \times 100 = \frac{W/\gamma_s}{W/\gamma_b} \times 100 = \frac{\gamma_b}{\gamma_s} \times 100 = \frac{\gamma_b}{G_{sb} \cdot \gamma_w} \times 100 \\ \%Voids &= 100 - \%V_s \end{aligned} \quad (5.14)$$

where

- V_s = volume of aggregate
- γ_s = unit weight of aggregate
- γ_b = bulk unit weight of aggregate
- γ_w = unit weight of water

Sample Problem 5.2

Coarse aggregate is placed in a rigid bucket and rodded with a tamping rod to determine its unit weight. The following data are obtained:

Volume of bucket = 1/3 ft³

Weight of empty bucket = 18.5 lb

Weight of bucket filled with dry rodded coarse aggregate = 55.9 lb

- a. Calculate the dry-rodded unit weight
- b. If the bulk dry specific gravity of the aggregate is 2.630, calculate the percent voids in the aggregate.

Solution

a. Dry-rodded unit weight = $(55.9 - 18.5)/0.333 = 112.3 \text{ lb/ft}^3$

b. Percent volume of particles = $\frac{112.3}{2.630 \times 62.3} \times 100 = 68.5\%$

Percent voids = $100 - 68.5 = 31.5\%$

5.5.7 ■ Strength and Modulus

The strength of portland cement concrete and asphalt concrete cannot exceed that of the aggregates. It is difficult and rare to test the strength of aggregate particles. However, tests on the parent rock sample or a bulk aggregate sample provide an indirect estimate of these values. Aggregate strength is generally important in high-strength concrete and in the surface course on heavily traveled pavements. The tensile strength of aggregates ranges from 0.7 MPa to 16 MPa (100 psi to 2300 psi), while the compressive strength ranges from 35 MPa to 350 MPa (5000 psi to 50,000 psi) (Meininger and Nichols 1990; Barksdale 1991). Field service records are a good indication of the adequacy of the aggregate strength.

The modulus of elasticity of aggregates is not usually measured. However, new mechanistic-based methods of pavement design require an estimate of the modulus of aggregate bases. The response of bulk aggregates to stresses is nonlinear and depends on the confining pressure on the material. Since the modulus is used for pavement design, dynamic loads are used in a test to simulate the magnitude and duration of stresses in a pavement base caused by a moving truck. During the test, as the stresses are applied to the sample, the deformation response has two components, a recoverable or resilient deformation, and a permanent deformation. Only the resilient portion of the strain is used with the applied stress level to compute the modulus of the aggregate. Hence, the results are defined as the resilient modulus M_R .

In the resilient modulus test (AASHTO T292), a prepared cylindrical sample is placed in a triaxial cell, as shown in Figure 5.9. A specimen with large aggregates is typically 0.15 m (6 in.) in diameter by 0.30 m (12 in.) high, while soil samples are 71 mm (2.8 in.) in diameter by 142 mm (5.6 in.) high. The specimen is subjected to a specified confining pressure and a repeated axial load. Accurate transducers, such as LVDTs, measure the axial deformation. The test requires a determination of the modulus over a range of axial loads and confining pressures. The resilient modulus equals the repeated axial stress divided by the resilient strain for each combination of load level and confining pressure. The resilient modulus test requires the measurement of very small loads and deformations and is, therefore, difficult to perform. Currently, the test is mostly limited to research projects.

5.5.8 ■ Gradation and Maximum Size

Gradation describes the particle size distribution of the aggregate. The particle size distribution is an important attribute of the aggregates. Large aggregates are economically advantageous in portland cement and asphalt concrete, as they have less surface area and, therefore, require less binder. However, large aggregate mixes, whether asphalt or portland cement concrete, are harsher and more difficult to work into place. Hence, construction considerations, such as equipment capability, dimensions of construction members, clearance between reinforcing steel, and layer thickness, limit the maximum aggregate size.

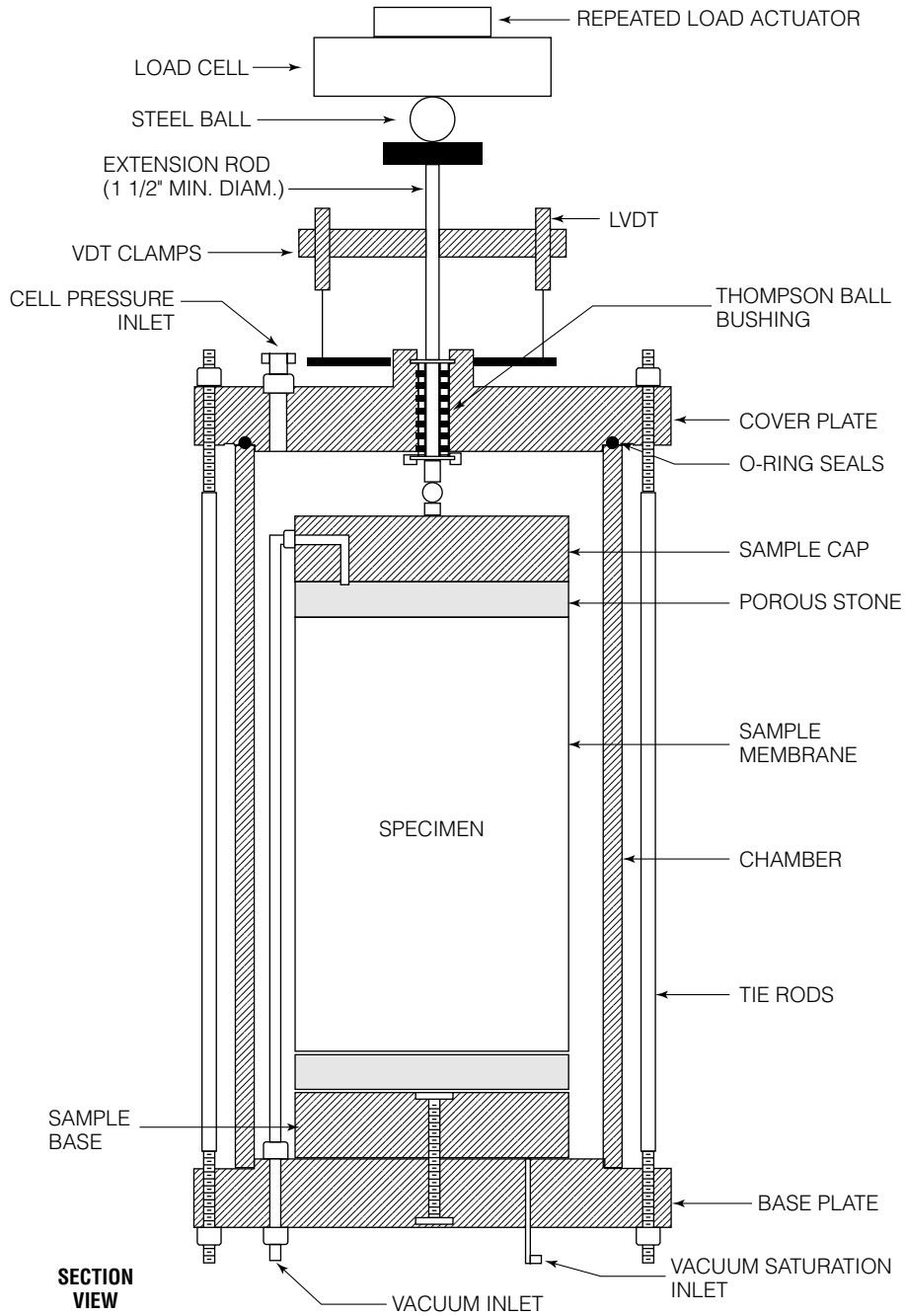


FIGURE 5.9 Triaxial chamber with external LVDT's and load cell.

Two definitions are used to describe the maximum particle size in an aggregate blend:

Maximum aggregate size—the smallest sieve size through which 100% of the aggregates sample particles pass.

Nominal maximum aggregate size—the largest sieve that retains any of the aggregate particles, but generally not more than 10%.

Some agencies define the maximum aggregate size as two sizes larger than the first sieve to retain more than 10% of the material, while the nominal maximum size is one size larger than the first sieve to retain more than 10% of the material (The Asphalt Institute 1995; McGennis et al. 1995).

Sieve Analysis Gradation is evaluated by passing the aggregates through a series of sieves, as shown in Figure 5.10 (ASTM C136, E11). The sieve retains particles larger than the opening, while smaller ones pass through. Metric sieve descriptions are based on the size of the openings measured in millimeters. Sieves smaller than 0.6 mm can be described in either millimeters or micrometers. In U.S. customary units, sieves with openings greater than 1/4 in. are designated by the size of the opening; the lengths of the sides of the square openings of a 2-in. sieve are 2 in. measured between the wires. This equals the diameter of a sphere that will exactly touch each side of the square at the midpoints. Sieves smaller than 1/4 in. are specified by the number of uniform openings per linear inch (a No. 8 sieve has 8 openings per inch, or 64 holes per square inch).

Gradation results are described by the cumulative percentage of aggregates that either pass through or are retained by a specific sieve size. Percentages are



FIGURE 5.10 Sieve shaker for large samples of aggregates.

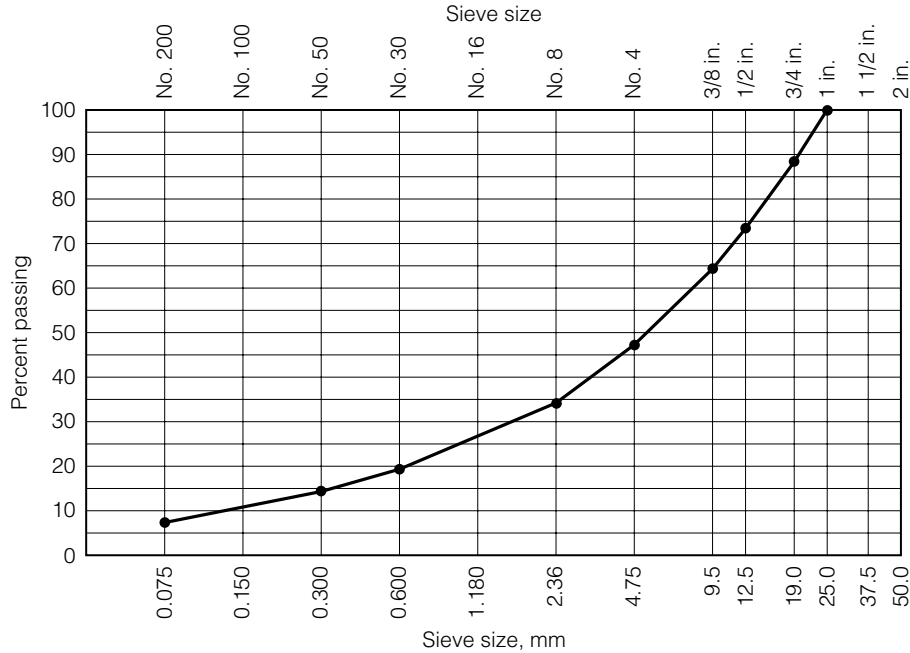


FIGURE 5.11 Semi-log aggregate gradation chart showing a gradation example. See Table 5.2.

reported to the nearest whole number, except that if the percentage passing the 0.075-mm (No. 200) sieve is less than 10%, it is reported to the nearest 0.1%. Gradation analysis results are generally plotted on a semilog chart, as shown in Figures 5.11 and A.21.

Aggregates are usually classified by size as coarse aggregates, fine aggregates, and mineral fillers (fines). ASTM defines coarse aggregate as particles retained on the 4.75-mm (No. 4) sieve, fine aggregate as those passing the 4.75-mm sieve, and mineral filler as material mostly passing the 0.075-mm (No. 200) sieve.

Maximum Density Gradation The density of an aggregate mix is a function of the size distribution of the aggregates. In 1907 Fuller established the relationship for determining the distribution of aggregates that provides the maximum density or minimum amount of voids as

$$P_i = 100 \left(\frac{d_i}{D} \right)^n \quad (5.15)$$

where

P_i = percent passing a sieve of size d_i

d_i = the sieve size in question

D = maximum size of the aggregate

The value of the exponent n recommended by Fuller is 0.5. In the 1960s, the Federal Highway Administration recommended a value of 0.45 for n and introduced the “0.45 power” gradation chart, Figures 5.12 and A.22, designed to produce a straight line for maximum density gradations (Federal Highway Administration 1988). Table 5.2 presents a sample

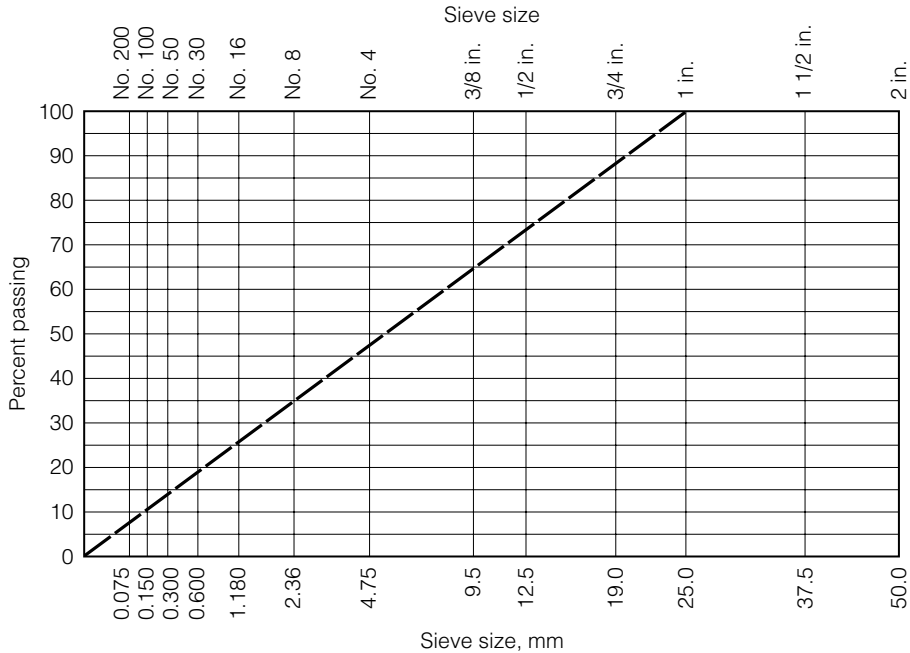


FIGURE 5.12 Federal Highway Administration 0.45 power gradation chart showing the maximum density gradation for a maximum size of 25 mm. See Table 5.2.

TABLE 5.2 Sample Calculations of Aggregate Distribution Required to Achieve Maximum Density

Sieve	$P_i = 100(d_i/D)^{0.45}$
25 mm (1 in.)	100
19 mm (3/4 in.)	88
12.5 mm (1/2 in.)	73
9.5 mm (3/8 in.)	64
4.75 mm (No. 4)	47
2.36 mm (No. 8)	34
0.60 mm (No. 30)	19
0.30 mm (No. 50)	14
0.075 mm (No. 200)	7.3

calculation of the particle size distribution required for maximum density. Note that the gradation in Table 5.2 is plotted on both gradation charts in Figures 5.11 and 5.12.

Frequently, a *dense* gradation, but not necessarily the maximum possible density, is desired in many construction applications, because of its high stability. Using a high-density gradation also means the aggregates occupy most of the volume of the material, limiting the binder content and thus reducing the cost. For example, aggregates for asphalt concrete must be dense, but must also have sufficient voids in the mineral aggregate to provide room for the binder, plus room for voids in the mixture.

Sample Problem 5.3

A sieve analysis test was performed on a sample of fine aggregate and produced the following results:

Sieve, mm	4.75	2.36	2.00	1.18	0.60	0.30	0.15	0.075	pan
Amount retained, g	0	33.2	56.9	83.1	151.4	40.4	72.0	58.3	15.6

Calculate the percent passing each sieve, and draw a 0.45 power gradation chart with the use of a spreadsheet program.

Solution

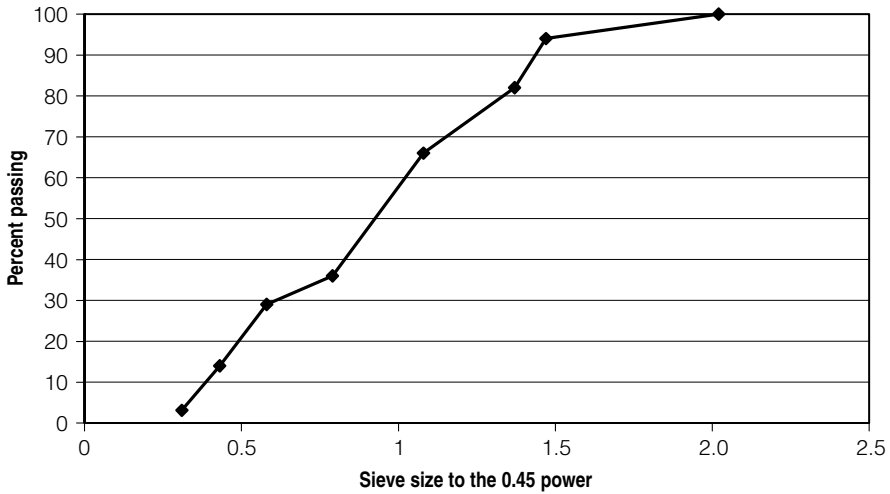
Sieve size	Amount Retained, g (a)	Cumulative Amount Retained, g (b)	Cumulative Percent Retained (c) = (b) × 100/Total	Percent Passing* (d) = 100 - (c)
4.75 mm (No. 4)	0	0	0	100
2.36 mm (No. 8)	33.2	33.2	6	94
2.00 mm (No. 10)	56.9	90.1	18	82
1.18 mm (No. 16)	83.1	173.2	34	66
0.60 mm (No. 30)	151.4	324.6	64	36
0.30 mm (No. 50)	40.4	365.0	71	29
0.15 mm (No. 100)	72.0	437.0	86	14
0.075 mm (No.200)	58.3	495.3	96.9	3.1
Pan	15.6	510.9	100	
Total	510.9			

*Percent passing is computed to a whole percent, except for the 0.075 mm (No. 200) material, which is computed to 0.1 %.

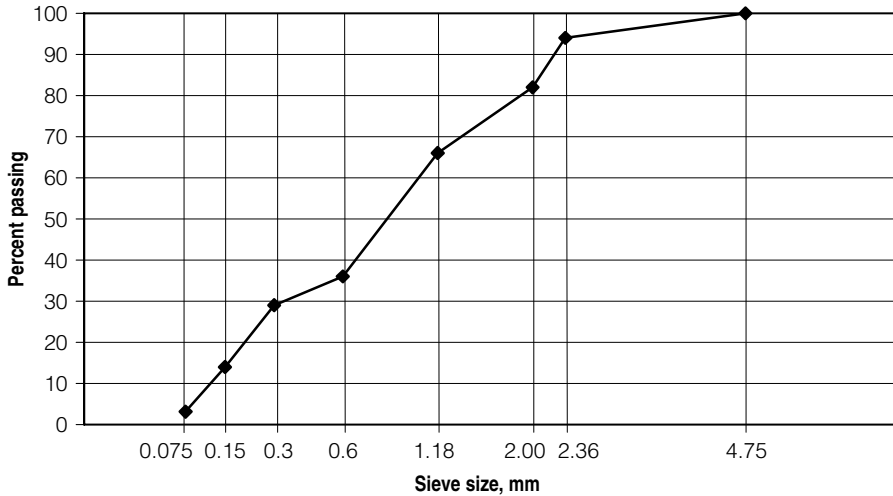
The first step in drawing the graph is to compute the sieve size to the 0.45 power, using the metric sieve sizes:

Sieve Size (mm)	Sieve to the 0.45 power	Percent Passing
4.75	2.02	100
2.36	1.47	94
2	1.37	82
1.18	1.08	66
0.6	0.79	36
0.3	0.58	29
0.15	0.43	14
0.075	0.31	3.1

Then the x-y scatter graph function is used to plot the percent passing on the y axis versus the sieve size to the 0.45 power:



Since the sieve size raised to the 0.45 power is not a meaningful number, the values on the axis are deleted and the text box feature is used to label the x-axis with the actual sieve values. In addition, the drawing tool is used to add vertical lines between the axis and the data points. The resulting graph is as follows:



Other Types of Gradation In addition to maximum density (i.e., *well-graded*), aggregates can have other characteristic distributions, as shown in Figure 5.13. A *one-sized* distribution has the majority of aggregates passing one sieve and being retained on the next smaller sieve. Hence, the majority of the aggregates have essentially the same diameter; their gradation curve is nearly vertical. One-sized graded aggregates will have good permeability, but poor stability, and are used in such applications as chip seals of pavements. *Gap-graded* aggregates are missing one or more sizes of material. Their gradation curve has a near horizontal section indicating that nearly the same portions of the aggregates pass two different sieve sizes. *Open-graded* aggregates are missing small aggregate sizes that would block the voids between the larger aggregate. Since there are a lot of voids, the material will be highly permeable, but may not have good stability.

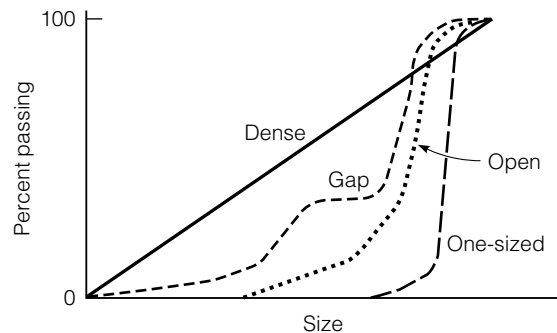


FIGURE 5.13 Types of aggregate grain size distributions plotted on a 0.45 gradation chart.

TABLE 5.3 Effect of Amount of Fines on the Relative Properties of Aggregate Base Material

Characteristic	No fines (Open or Clean)	Well-Graded (Dense)	Large Amount of Fines (Dirty or Rich)
Stability	Medium	Excellent	Poor
Density	Low	High	Low
Permeability	Permeable	Low	Impervious
Frost Susceptibility	No	Maybe	Yes
Handling	Difficult	Medium	Easy
Cohesion	Poor	Medium	Large

As shown in Table 5.3, the amount of fines has a major effect on the characteristics of aggregate base materials. Aggregates with the percentage of fines equal to the amount required for maximum density have excellent stability and density, but may have a problem with permeability, frost susceptibility, handling, and cohesion.

Gradation Specifications Gradation specifications define maximum and minimum cumulative percentages of material passing each sieve. Aggregates are commonly described as being either coarse or fine, depending on whether the material is predominantly retained on or passes through a 4.75-mm (No. 4) sieve.

Portland cement concrete requires separate specifications for coarse and fine aggregates. The ASTM C33 specifications for fine aggregates for concrete are given in Table 5.4. Table 5.5 shows the ASTM C33 gradation specifications for coarse concrete aggregates.

TABLE 5.4 ASTM Gradation Specifications for Fine Aggregates for Portland Cement Concrete (Copyright ASTM, reprinted with permission)

Sieve	Percent Passing
9.5 mm (3/8")	100
4.75 mm (No. 4)	95–100
2.36 mm (No. 8)	80–100
1.18 mm (No. 16)	50–85
0.60 mm (No. 30)	25–60
0.30 mm (No. 50)	10–30
0.15 mm (No. 100)	2–10

TABLE 5.5 Coarse Aggregate Grading Requirements for Concrete (ASTM C-33) (Copyright ASTM, reprinted with permission)

Amounts Finer Than Each Laboratory Sieve (Square Openings), Weight Percent														
Size No.	Nominal Size	4 in. (100 mm)	3 1/2 in. (90 mm)	3 in. (75 mm)	2 1/2 in. (63 mm)	2 in. (50 mm)	1 1/2 in. (37.5 mm)	1 in. (25.0 mm)	3/4 in. (19.0 mm)	1/2 in. (12.5 mm)	3/8 in. (9.5 mm)	No. 4 (4.75 mm)	No. 8 (2.36 mm)	No. 16 (1.18 mm)
1	3 1/2 to 1 1/2 in. (90 to 37.5 mm)	100	90 to 100	...	25 to 60	...	0 to 15	...	0 to 5
2	2 1/2 to 1 1/2 in. (63 to 37.5 mm)	100	90 to 100	35 to 70	0 to 15	...	0 to 5
3	2 to 1 in. (50 to 25.0 mm)	100	90 to 100	35 to 70	0 to 15	...	0 to 5
357	2 in. to No. 4 (50 to 4.75 mm)	100	95 to 100	...	35 to 70	...	10 to 30	...	0 to 5
4	3/4 in. (19 mm)	100	90 to 100	20 to 55	0 to 15	...	0 to 5
467	1 1/2 in. to No. 4 (37.5 to 4.75 mm)	100	95 to 100	...	35 to 70	...	10 to 30	0 to 5

TABLE 5.5 (Continued)

Amounts Finer Than Each Laboratory Sieve (Square Openings), Weight Percent

Size No.	Nominal Size	4 in. (100 mm)	3 1/2 in. (90 mm)	3 in. (75 mm)	2 1/2 in. (63 mm)	2 in. (50 mm)	1 1/2 in. (37.5 mm)	1 in. (25.0 mm)	3/4 in. (19.0 mm)	1/2 in. (12.5 mm)	3/8 in. (9.5 mm)	No. 4 (4.75 mm)	No. 8 (2.36 mm)	No. 16 (1.18 mm)
5	1 to 1/2 in. (25.0 to 12.5 mm)	100	90 to 100	20 to 55	0 to 10	0 to 5
56	1 to 3/8 in. (25.0 to 9.5 mm)	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5
57	1 in. to No. 4 (25.0 to 4.75 mm)	100	95 to 100	...	25 to 60	...	0 to 10	0 to 5	...
6	3/4 in. to 3/8 in. (19.0 to 9.5 mm)	100	90 to 100	20 to 55	0 to 15	0 to 5
67	3/4 in. to No. 4 (19.0 to 4.75 mm)	100	90 to 100	...	20 to 55	0 to 10	0 to 5	...
7	1/2 in. to No. 4 (12.5 to 4.75 mm)	100	90 to 100	40 to 70	0 to 15	0 to 5	...
8	3/8 in. to No. 8 (9.5 to 2.36 mm)	100	85 to 100	10 to 30	0 to 10	0 to 5

TABLE 5.6 Aggregate Grading Requirements for Superpave Hot Mix Asphalt (AASHTO MP-2)

Sieve Size, mm (in.)	Nominal Maximum Size (mm)					
	37.5	25	19	12.5	9.5	4.75
50 (2 in.)	100	—	—	—	—	—
37.5 (1 1/2 in.)	90–100	100	—	—	—	—
25 (1 in.)	90 max	90–100	100	—	—	—
19 (3/4 in.)	—	90 max	90–100	100	—	—
12.5 (1/2 in.)	—	—	90 max	90–100	100	100
9.5 (3/8 in.)	—	—	—	90 max	90–100	95–100
4.75 (No. 4)	—	—	—	—	90 max	90–100
2.36 (No. 8)	15–41	19–45	23–49	28–58	32–67	—
1.18 (No. 16)	—	—	—	—	—	30–60
0.075 (No. 200)	0.0–6.0	1.0–7.0	2.0–8.0	2.0–10.0	2.0–10.0	6.0–12.0

Generally, local agencies develop their own specifications for the gradation of aggregates for asphalt concrete. Table 5.6 gives the aggregate grading requirements for Superpave hot mix asphalt (McGennis et al. 1995). These specifications define the range of allowable gradations for asphalt concrete for mix design purposes. Note that the percentage of material passing the 0.075-mm (No. 200) sieve, the fines or mineral filler, is carefully controlled for asphalt concrete due to its significance to the properties of the mix.

Once aggregate gradation from asphalt concrete mix design is established for a project, the contractor must produce aggregates that fall within a narrow band around the single gradation line established for developing the mix design. For example, the Arizona Department of Transportation will give the contractor full pay only if the gradation of the aggregates is within the following limits with respect to the accepted mix design gradations:

Sieve Size	Allowable Deviations for Full Pay
9.5 mm (3/8 in.) and larger	±3%
2.36 to 0.45 mm (No. 8 to No. 40)	±2%
0.075 mm (No. 200)	±0.5%

TABLE 5.7 Sample Calculation of Fineness Modulus

Sieve Size	Percentage of Individual Fraction Retained, by Weight	Cumulative Percentage Retained by Weight	Percentage Passing by Weight
9.5 mm (3/8 in.)	0	0	100
4.75 mm (No. 4)	2	2	98
2.36 mm (No. 8)	13	15	85
1.18 mm (No. 16)	25	40	60
0.60 mm (No. 30)	15	55	45
0.30 mm (No. 50)	22	77	23
0.15 mm (No. 100)	20	97	3
pan	3	100	0
Total	100		

$$\text{Fineness Modulus} = 286/100 = 2.86$$

Fineness Modulus The *fineness modulus* is a measure of the fine aggregates' gradation and is used primarily for portland cement concrete mix design. It can also be used as a daily quality control check in the production of concrete. The fineness modulus is one-hundredth of the sum of the cumulative percentage weight retained on the 0.15-mm, 0.3-mm, 0.6-mm, 1.18-mm, 2.36-mm, 4.75-mm, 9.5-mm, 19.0-mm, 37.5-mm, 75-mm, and 150-mm (No. 100, 50, 30, 16, 8, and 4 and 3/8-in., 3/4-in., 1½-in., 3-in., and 6-in.) sieves. When the fineness modulus is determined for fine aggregates, sieves larger than 9.5 mm (3/8 in.) are not used. The fineness modulus should be in the range of 2.3 to 3.1, with a higher number being a coarser aggregate. Table 5.7 demonstrates the calculation of the fineness modulus.

Sample Problem 5.4

Calculate the fineness modulus of the sieve analysis results of sample problem 5.1.

Solution

According to the definition of fineness modulus, sieves 2.00 and 0.075 mm (No. 10 and 200) are not included.

$$\text{Fineness modulus} = \frac{6 + 34 + 64 + 71 + 86}{100} = 2.61$$

Blending Aggregates to Meet Specifications Generally, a single aggregate source is unlikely to meet gradation requirements for portland cement or asphalt concrete mixes. Thus, blending of aggregates from two or more sources would be required to satisfy the specifications. Figure 5.14 shows a graphical method for selecting the combination of two aggregates to meet a specification. Table 5.8 presents the data used for Figure 5.14. Determining a satisfactory aggregate blend with the graphical method entails the following steps (The Asphalt Institute 1995):

1. Plot the percentages passing through each sieve on the right axis for aggregate A and on the left axis for aggregate B, shown as open circles in Figure 5.14.
2. For each sieve size, connect the left and right axes.
3. Plot the specification limits of each sieve on the corresponding sieve lines; that is, a mark is placed on the 9.5-mm (3/8 in.) sieve line corresponding to 70% and 90% on the vertical axis, shown as closed circles in Figure 5.14.
4. Connect the upper- and lower-limit points on each sieve line.
5. Draw vertical lines through the rightmost point of the upper-limit line and the leftmost point of the lower-limit line. If the upper- and lower-limit lines overlap, no combination of the aggregates will meet specifications.
6. Any vertical line drawn between these two vertical lines identifies an aggregate blend that will meet the specification. The intersection with the upper axis defines the percentage of aggregate B required for the blend. The projection to the lower axis defines the percentage of aggregate A required.
7. Projecting intersections of the blend line and the sieve lines horizontally gives an estimate of the gradation of the blended aggregate. Figure 5.14 shows that a 50-50 blend of aggregates A and B will result in a blend with 43% passing through the 2.36-mm (No. 8) sieve. The gradation of the blend is shown in the last line of Table 5.8.

When more than two aggregates are required, the graphical procedure can be repeated in an iterative manner. However, a trial and error process is generally used to determine the proportions. The basic equation for blending is

$$P_i = Aa + Bb + Cc + \dots \quad (5.16)$$

where

$$\begin{aligned} P_i &= \text{percent blend material passing sieve size } i \\ A, B, C, \dots &= \text{percent of aggregates A, B, C, } \dots \text{ passing sieve } i \\ a, b, c, \dots &= \text{decimal fractions by weight of aggregates A, B, and C} \\ &\quad \text{used in the blend, where the total is 1.00} \end{aligned}$$

Table 5.9 demonstrates these calculations for two aggregate sources. The table shows the required specification range and the desired (or target)

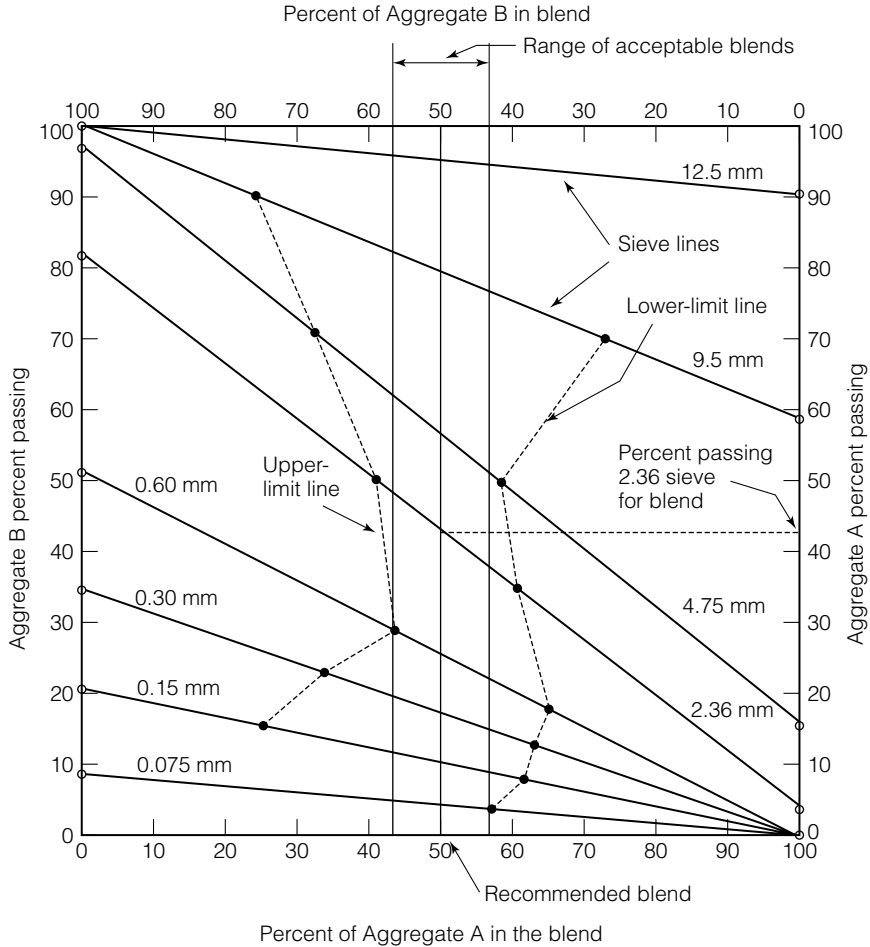


FIGURE 5.14 Graphical method for determining aggregate blend to meet gradation requirements. See Table 5.8.

TABLE 5.8 Example of Aggregate Blending Analysis by Graphical Method

Sieve	19 mm (3/4 in.)	12.5mm (1/2 in.)	9.5mm (3/8 in.)	4.75mm (No. 4)	2.36mm (No. 8)	0.60mm (No. 30)	0.30mm (No. 50)	0.15 mm (No.100)	0.075mm (No.200)
Speci- fication	100	80–100	70–90	50–70	35–50	18–29	13–23	8–16	4–10
Agg. A	100	90	59	16	3	0	0	0	0
Agg. B	100	100	100	96	82	51	36	21	9
Blend	100	95	80	56	43	26	18	11	4.5

Note: Numbers shown are percent passing each sieve.

TABLE 5.9 Example of Aggregate Blending Analysis by Iterative Method.

Sieve	12.5 mm (1/2 in.)	9.5 mm (3/8 in.)	4.75 mm (No. 4)	2.00 mm (No. 10)	0.425 mm (No. 40)	0.180 mm (No. 80)	0.075 mm (No. 200)
Specification	100	95–100	70–85	55–70	20–40	10–20	4–8
Target gradation	100	98	77.5	62.5	30	15	6
% Agg. A (A)	100	100	98	90	71	42	19
% Agg. B (B)	100	94	70	49	14	2	1
30% A (a)	30	30	29.4	27	21.3	12.6	5.7
70% B (b)	70	65.8	49	34.3	9.8	1.4	0.7
Blend (P_i)	100	96	78	61	31	14	6.4

gradation, usually the midpoint of the specification. A trial percentage of each aggregate source is assumed and is multiplied by the percentage passing each sieve. These gradations are added to get the composite percentage passing each sieve for the blend. The gradation of the blend is compared to the specification range to determine if the blend is acceptable. With practice, blends of four aggregates can readily be resolved. These calculations are easily performed by a spreadsheet computer program.

Properties of Blended Aggregates When two or more aggregates from different sources are blended, some of the properties of the blend can be calculated from the properties of the individual components. With the exception of specific gravity and density, the properties of the blend are the simple weighted averages of the properties of the components. This relationship can be expressed as

$$X = P_1X_1 + P_2X_2 + P_3X_3 + \dots \quad (5.17a)$$

where

$$\begin{aligned} X &= \text{composite property of the blend} \\ X_1, X_2, X_3 &= \text{properties of fractions 1, 2, 3} \\ P_1, P_2, P_3 &= \text{decimal fractions by weight of aggregates 1, 2, 3 used in} \\ &\quad \text{the blend, where the total is 1.00} \end{aligned}$$

This equation applies to properties such as angularity, absorption, strength, and modulus.

Sample Problem 5.5

Coarse aggregates from two stockpiles having coarse aggregate angularity (crushed faces) of 40% and 90% were blended at a ratio of 30:70 by weight, respectively. What is the percent of crushed faces of the aggregate blend?

Solution

$$\text{Crushed faces of the blend} = (0.3)(40) + (0.7)(90) = 75\%$$

Equation 5.17a is used for properties that apply to the whole aggregate materials in all stockpiles that are blended. However, some properties apply to either coarse aggregate only or fine aggregate only. Therefore, the percentage of coarse or fine aggregate in each stockpile has to be considered. The relationship in this case is expressed as

$$X = \frac{(x_1P_1p_1 + x_2P_2p_2 + \cdots + x_nP_np_n)}{(P_1p_1 + P_2p_2 + \cdots + P_np_n)} \quad (5.17b)$$

where,

X = the test value for the aggregate blend

x_i = the test result for stockpile i

P_i = the percent of stockpile i in the blend

p_i = the percent of stockpile i that either passes or is retained on the dividing sieve

Sample Problem 5.6

Aggregates from two stockpiles, A and B having coarse aggregate angularity (crushed faces) of 40% and 90% were blended at a ratio of 30:70 by weight, respectively. The percent material passing the 4.75 mm sieve was 25% and 55% for stockpiles A and B, respectively. What is the percent of crushed faces of the aggregate blend?

Solution

Crushed faces of the blend =

$$\begin{aligned} X &= \frac{(x_1P_1p_1 + x_2P_2p_2 + \cdots + x_nP_np_n)}{(P_1p_1 + P_2p_2 + \cdots + P_np_n)} \\ &= \frac{(40 \times 30 \times (100 - 25) + 90 \times 70 \times (100 - 55))}{(30 \times (100 - 25) + 70 \times (100 - 55))} = 69\% \end{aligned}$$

Note that the percentage of coarse aggregate in each stockpile was calculated by subtracting the percentage passing the 4.75 mm sieve from 100.

Asphalt concrete mix design requires that the engineer knows the composite specific gravity of all aggregates in the mix. The composite specific gravity of a mix of different aggregates is obtained by the formula

$$G = \frac{1}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \frac{P_3}{G_3} + \dots} \quad (5.18)$$

where

- G = composite specific gravity
- G_1, G_2, G_3 = specific gravities of fractions 1, 2, and 3
- P_1, P_2, P_3 = decimal fractions by weight of aggregates 1, 2, and 3 used in the blend, where the total is 1.00

Note that Equation 5.18 is used only to obtain the combined specific gravity and density of the blend, whereas Equation 5.17 is used to obtain other combined properties.

Sample Problem 5.7

Aggregates from three sources having bulk specific gravities of 2.753, 2.649, and 2.689 were blended at a ratio of 70:20:10 by weight, respectively. What is the bulk specific gravity of the aggregate blend?

Solution

$$G = \frac{1}{\frac{0.7}{2.753} + \frac{0.2}{2.649} + \frac{0.1}{2.689}} = 2.725$$

5.5.9 Deleterious Substances in Aggregate

A deleterious substance is any material that adversely affects the quality of portland cement or asphalt concrete made with the aggregate. Table 5.10 identifies the main deleterious substances in aggregates and their effects on portland cement concrete. In asphalt concrete, deleterious substances are clay lumps, soft or friable particles, and coatings. These substances decrease the adhesion between asphalt and aggregate particles.

TABLE 5.10 Main Deleterious Substances and Their Affects on Portland Cement Concrete

Substance	Harmful Effect
Organic impurities	Delay setting and hardening, may reduce strength gain, may cause deterioration
Minus 0.075 mm (No. 200) materials	Weaken bond, may increase water requirements
Coal, lignite or other low-density materials	Reduce durability, may cause popouts or stains
Clay lumps and friable particles	Popouts, reduce durability and wear resistance
Soft particles	Reduce durability and wear resistance, popouts

5.5.10 ■ Alkali–Aggregate Reactivity

Some aggregates react with portland cement, harming the concrete structure. The most common reaction, particularly in humid and warm climates, is between the active silica constituents of an aggregate and the alkalis in cement (sodium oxide, Na_2O , and potassium oxide, K_2O). The alkali–silica reaction results in excessive expansion, cracking, or popouts in concrete as shown in Figure 5.15. Other constituents in the aggregate, such as carbonates, can also react with the alkali in the cement; however, their reaction is less harmful. The alkali–aggregate reactivity is affected by the amount, type, and particle size of the reactive material, as well as by the soluble alkali and water content of the concrete.

The best way to evaluate the potential for alkali–aggregate reactivity is by reviewing the field service history. For aggregates without field service history, several laboratory tests are available to check the potential alkali–aggregate reactivity. The ASTM C227 test can be used to determine the potentially expansive alkali–aggregate reactivity of cement–aggregate combinations. In this test, a mortar bar is stored under a prescribed temperature and moisture conditions and its expansion is determined. The quick chemical test (ASTM C289) can be used to identify potentially reactive siliceous aggregates. ASTM C586 is used to determine potentially expansive carbonate rock aggregates (alkali–carbonate reactivity).

If alkali-reactive aggregate must be used, the reactivity can be minimized by limiting the alkali content of the cement. The reactivity can also be reduced by keeping the concrete structure as dry as possible. Fly ash, ground granulated blast furnace slag, silica fume, or natural pozzolans can be used to control the alkali–silica reactivity. Lithium-based admixtures have also been used for the same purpose. Finally, replacing about 30% of a reactive

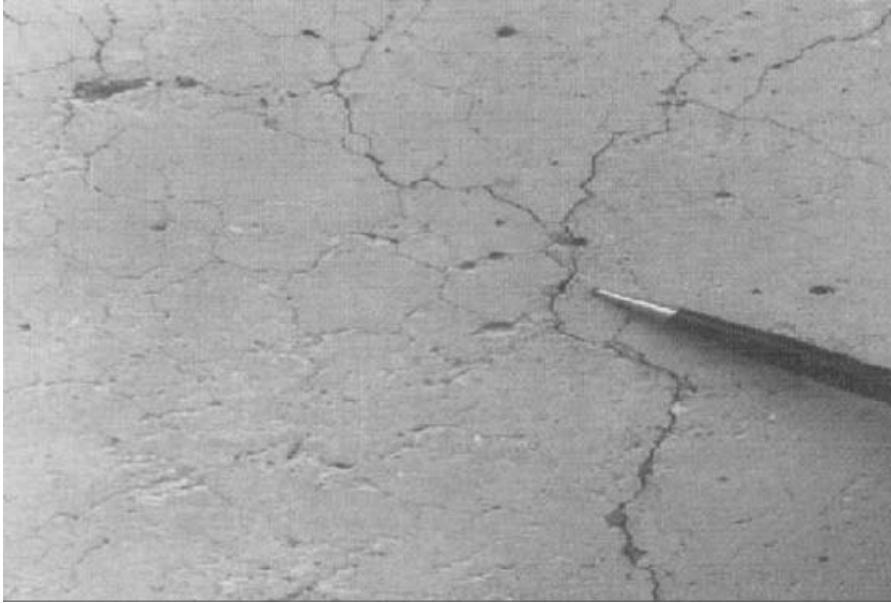


FIGURE 5.15 Example of cracking in concrete due to alkali-silica reactivity.

sand-gravel aggregate with crushed limestone (limestone sweetening) can minimize the alkali reactivity (Kosmatka et al. 2002).

5.5.11 ■ Affinity for Asphalt

Stripping, or moisture-induced damage, is a separation of the asphalt film from the aggregate through the action of water, reducing the durability of the asphalt concrete and resulting in pavement failure. The mechanisms causing stripping are complex and not fully understood. One important factor is the relative affinity of the aggregate for either water or asphalt. *Hydrophilic* (water-loving) aggregates, such as silicates, have a greater affinity for water than for asphalt. They are usually acidic in nature and have a negative surface charge. Conversely, *hydrophobic* (water-repelling) aggregates have a greater affinity for asphalt than for water. These aggregates, such as limestone, are basic in nature and have a positive surface charge. Hydrophilic aggregates are more susceptible to stripping than hydrophobic aggregates. Other stripping factors include porosity, absorption, and the existence of coatings and other deleterious substances.

Since stripping is the result of a compatibility problem between the asphalt and the aggregate, tests for stripping potential are performed on the asphalt concrete mix. Early compatibility tests submerged the sample in either room-temperature water (ASTM D1664) or boiling water (ASTM D3625); after a period of time, the technician observed the percentage of particles stripped from the asphalt. More recent procedures subject asphalt concrete

to cycles of freeze–thaw conditioning. The strength or modulus of the specimens is measured and compared with the values of unconditioned specimens (ASTM D1075).

5.6 Handling Aggregates

Aggregates must be handled and stockpiled in such a way as to minimize segregation, degradation, and contamination. If aggregates roll down the slope of the stockpile, the different sizes will segregate, with large stones at the bottom and small ones at the top. Building stockpiles in thin layers circumvents this problem. The drop height should be limited to avoid breakage, especially for large aggregates. Vibration and jiggling on a conveyor belt tends to work fine material downward while coarse particles rise. Segregation can be minimized by moving the material on the belt frequently (up and down, side to side, in and out) or by installing a baffle plate, rubber sleeve, or paddle wheel at the end of the belt to remix coarse and fine particles. Rounded aggregates segregate more than crushed aggregates. Also, large aggregates segregate more readily than smaller aggregates. Therefore, different sizes should be stockpiled and batched separately. Stockpiles should be separated by dividers or placed in bins to avoid mixing and contamination (Figure 5.16) (Meininger and Nichols 1990).

5.6.1 Sampling Aggregates

In order for any of the tests described in this chapter to be valid, the sample of material being tested must represent the whole population of materials that is being quantified with the test. This is a particularly difficult problem with aggregates due to potential segregation problems. Samples of aggregates can be collected from any location in the production process, that is, from the stockpile, conveyor belts, or from bins within the mixing machinery (ASTM D75). Usually, the best location for sampling the aggregate is on the conveyor belt that feeds the mixing plant. However, since the aggregate segregates on the belt, the entire width of the belt should be sampled at several locations or times throughout the production process. The samples would then be mixed to represent the entire lot of material.



FIGURE 5.16 Aggregate bins used to stockpile aggregates with different sizes.

Sampling from stockpiles must be performed carefully to minimize segregation. Typically, aggregate samples are taken from the top, middle, and bottom of the stockpile and then combined. Before taking the samples, discard the 75mm to 150mm (3 in. to 6 in.) material at the surface. A board shoved vertically into the pile just above the sampling point aids in preventing rolling of coarse aggregates during sampling. Samples are collected using a square shovel and are placed in sample bags or containers and labeled.

Sampling tubes 1.8 m (6 ft) long and 30 mm (1.25 in.) in diameter are used to sample fine aggregate stockpiles. At least five samples should be collected from random locations in the stockpile. These samples are then combined before laboratory testing.

Field sample sizes are governed by the nominal maximum size of aggregate particles (ASTM D75). Larger-sized aggregates require larger samples to minimize segregation errors. Field samples are typically larger than the samples needed for testing. Therefore, field samples must be reduced using sample splitters (Figure 5.17) or by quartering (Figure 5.18) (ASTM C702).



FIGURE 5.17 Aggregate sample splitter.

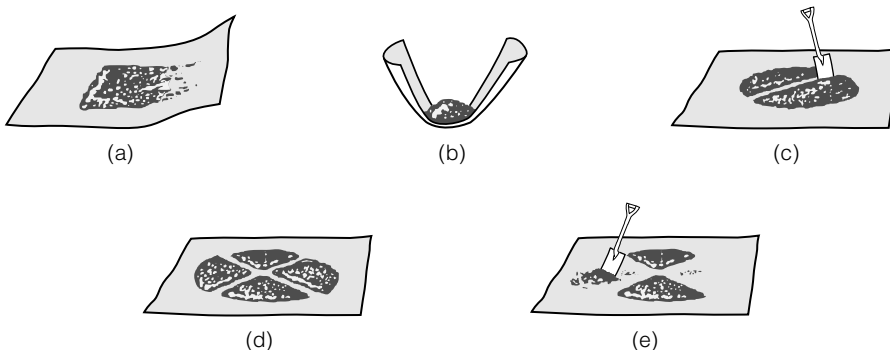


FIGURE 5.18 Steps for reducing the sample size by quartering: (a) mixing by rolling on blanket, (b) forming a cone after mixing, (c) flattening the cone and quartering, (d) finishing quartering, (e) retaining opposite quarters (the other two quarters are rejected). (ASTM C702). Copyright ASTM. Reprinted with permission.

S U M M A R Y

Aggregates are widely used as a base material for foundations and as an ingredient in portland cement concrete and asphalt concrete. While the geological classification of aggregates gives insight into the properties of the material, the suitability of a specific source of aggregates for a particular application requires testing and evaluation. The most significant attributes of aggregates include the gradation, specific gravity, shape and texture, and soundness. When used in concrete, the compatibility of the aggregate and the binder must be evaluated.

Q U E S T I O N S A N D P R O B L E M S

- 5.1 What are the three mineralogical or geological classifications of rocks and how are they formed?
- 5.2 Discuss five different desirable characteristics of aggregate used in portland cement concrete.
- 5.3 Discuss five different desirable characteristics of aggregate used in asphalt concrete.
- 5.4 The shape and surface texture of aggregate particles are important for both portland cement concrete and hot mix asphalt.
 - a. For preparing PCC, would you prefer round and smooth aggregate or rough and angular aggregate? Briefly explain why (no more than two lines).
 - b. For preparing HMA, would you prefer round and smooth aggregate or rough and angular aggregate? Briefly explain why (no more than two lines).
- 5.5 A sample of fine aggregate has the following properties:
Wet mass = 521.0 g
Dry mass = 491.6 g
Absorption = 2.5%
Determine: (a) total moisture content, and (b) free moisture content
- 5.6 Use the following information to determine the total and free moisture contents in percent:
Mass of wet sand = 627.3 g
Mass of dry sand = 590.1 g
Absorption = 1.5%

- 5.7 A sample of wet aggregate weighed 19.682 N. After drying this sample in an oven it weighed 18.365 N. The absorption of this aggregate is 4.8%. Calculate the percent of free water in the original wet sample.
- 5.8 Samples of coarse aggregate from a stockpile are brought to the laboratory for determination of specific gravities. The following weights are found:
- Mass of moist aggregate sample as brought to the laboratory:
5298 grams
 - Mass of oven dried aggregate: 5216 g
 - Mass of aggregates submerged in water: 3295 g
 - Mass of SSD (Saturated Surface Dry) Aggregate: 5227 g
- Find
- a. The aggregate bulk specific gravity
 - b. The aggregate apparent specific gravity
 - c. The moisture content of stockpile aggregate (report as a percent)
 - d. Absorption (report as percent)
- 5.9 Base course aggregate has a target dry density of 124.9 lb/cu ft in place. It will be laid down and compacted in a rectangular street repair area of 1000 ft \times 52 ft \times 6 in. The aggregate in the stockpile contains 2.7 percent moisture. If the required compaction is 95 percent of the target, how many tons of aggregate will be needed?
- 5.10. Calculate the percent voids between aggregate particles, which have been compacted by rodding, if the dry-rodded unit weight is 88.0 lb/cu ft and the bulk dry specific gravity is 2.701.
- 5.11. Coarse aggregate is placed in a rigid bucket and rodded with a tamping rod to determine its unit weight. The following data are obtained:
- Volume of bucket = $\frac{1}{2}$ ft³
 - Weight of empty bucket = 20.3 lb
 - Weight of bucket filled with dry rodded coarse aggregate = 69.6 lb
- a. Calculate the dry-rodded unit weight
 - b. If the bulk dry specific gravity of the aggregate is 2.620, calculate the percent voids between aggregate particles.
- 5.12 The following laboratory tests are performed on aggregate samples:
- a. Specific gravity and absorption
 - b. Soundness
 - c. Sieve analysis test
- What are the significance and use of each of these tests?
- 5.13 The specific gravity and absorption test (ASTM C128) was performed on fine aggregate and the following data were obtained:
- Mass of SSD sand = 500.0 g
 - Mass of pycnometer with water only = 623.0 g
 - Mass of pycnometer with sand and water = 938.2 g
 - Mass of dry sand = 495.5 g
- Calculate the specific gravity values (dry bulk, SSD, and apparent) and the absorption of the fine aggregate.

- 5.14 Referring to ASTM specification C33, what is the maximum sieve size and the nominal maximum sieve size for each of the standard sizes Numbers 4, 57, and 7.
- 5.15 Calculate the sieve analysis of the following aggregate and plot on a semilog gradation paper:

Sieve size	Amount Retained, g	Cumulative Amount Retained, g	Cumulative Percent Retained	Percent Passing
25 mm (1 in.)	0			
9.5 mm (3/8 in.)	35.2			
4.75 mm (No. 4)	299.6			
2.00 mm (No. 10)	149.7			
0.425 mm (No. 40)	125.8			
0.075 mm (No.200)	60.4			
Pan	7.3			

- 5.16 Calculate the sieve analysis of the following aggregate, and plot on a 0.45 power gradation chart:

Sieve size	Amount, g	Cumulative Amount Retained, g	Cumulative Percent Retained	Percent Passing
Plus 37.5 mm	0			
37.5 mm to 25 mm	206			
25 mm to 19 mm	603			
19 mm to 9.5 mm	1413			
9.5 mm to 4.75 mm	508			
4.75 mm to 0.60 mm	963			
0.60 mm to 0.075 mm	1425			
Pan	32			

- 5.17 A sieve analysis test was performed on a sample of aggregate and produced the following results:

Sieve Size, mm	Amount Retained, g	Sieve Size, mm	Amount Retained, g
25	0	1.18	891.5
19	376.7	0.60	712.6
12.5	888.4	0.30	625.2
9.5	506.2	0.15	581.5
4.75	1038.4	0.075	242.9
2.36	900.1	Pan	44.9

Calculate the percent passing through each sieve. Plot the percent passing versus sieve size on

- a. a semilog gradation chart, and
 - b. a 0.45 gradation chart (Figure A.22).
- 5.18 A sieve analysis test was performed on a sample of coarse aggregate and produced the following results:

Sieve Size	Amount Retained, lb
3 in.	0
2 in.	0
1-1/2 in.	5.2
1 in.	18.1
3/4 in.	14.8
1/2 in.	16.3
3/8 in.	25.0
No. 4	8.5
Pan	1.6

- a. Calculate the percent passing through each sieve.
 - b. What is the maximum size?
 - c. What is the nominal maximum size?
 - d. Plot the percent passing versus sieve size on a semilog gradation chart.
 - e. Plot the percent passing versus sieve size on a 0.45 gradation chart (Figure A.22).
 - f. Referring to ASTM C33, what is the closest size number and does it meet the gradation for that standard size?
- 5.19 Draw a graph to show the cumulative percent passing through the sieve versus sieve size for well-graded, gap-graded, open-graded, and one-sized aggregates.
- 5.20 Table P5.20 shows the grain size distributions of aggregates A, B, and C. The three aggregates must be blended at a ratio of 15:25:60 by weight, respectively. Using a spreadsheet program, determine the grain size distribution of the blend.

Table P5.20

	Percent Passing								
	Size								
	25 mm	19 mm	12.5 mm	9.5 mm	4.75 mm	1.18 mm	0.60 mm	0.30 mm	0.15 mm
Aggregate A	100	100	100	83	67	49	37	25	18
Aggregate B	100	100	74	51	32	24	19	13	7
Aggregate C	100	82	66	42	27	14	5	0	0

5.21 Table P5.21 shows the grain size distributions of two aggregates A and B.

Table P5.21

Sieve Size, mm	25	19	12.5	9.5	4.750	2.36	1.18	0.600	0.300	0.150	0.075
% Passing Agg. A	100	92	76	71	53	38	32	17	10	5	3.0
% Passing Agg. B	100	100	92	65	37	31	30	29	28	21	15.4

Answer the following questions and show all calculations:

- a. What are the maximum sizes of aggregates A and B?
- b. Is aggregate A well graded? Why?
- c. Is aggregate B well graded? Why?

5.22 Three aggregates are to be mixed together in the following ratio:

- Aggregate A 20%
- Aggregate B 45%
- Aggregate C 35%

For each aggregate, the percent passing a set of five sieves is shown in the following table:

Sieve Size (mm)	% Passing Agg. A	% Passing Agg. B	% Passing Agg. C
9.5	85	50	40
4.75	70	35	30
0.6	35	20	5
0.3	25	13	1
0.15	17	7	0

Determine the percent passing each sieve for the blended aggregate.

5.23 Table P5.23 shows the grain size distribution for two aggregates and the specification limits for an asphalt concrete. Determine the blend proportion required to meet the specification and the gradations of the blend. On a semilog gradation graph, plot the gradations of aggregate A, aggregate B, the selected blend, and the specification limits.

Table P5.23

	Percent Passing								
	Size								
	19 mm (3/4 in.)	12.5 mm (1/2 in.)	9.5 mm (3/8 in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	0.60 mm (No. 30)	0.30 mm (No. 50)	0.15 mm (No. 100)	0.075 mm (No. 200)
Spec. limits	100	80–100	70–90	50–70	35–50	18–29	13–23	8–16	4–10
Aggregate A	100	85	55	20	2	0	0	0	0
Aggregate B	100	100	100	85	67	45	32	19	11

5.24 Laboratory specific gravity and absorption tests are run on two coarse aggregate sizes, which have to be blended. The results are as follows:

- Aggregate A: Buck specific gravity = 2.814; absorption = 0.4%
- Aggregate B: Buck specific gravity = 2.441; absorption = 5.2%

- a. What is the specific gravity of a mixture of 50% aggregate A and 50% aggregate B by weight?
 - b. What is the absorption of the mixture?
- 5.25 The mix design for an asphalt concrete mixture requires 2 to 6 percent minus No. 200. The following three aggregates are available:

	Minus No. 200
Coarse	0.5%
Intermediate	1.5%
Fine Aggregate	11.5%

Considering that approximately equal amounts of coarse and intermediate aggregate will be used in the mix, what is the percentage of fine aggregate that will give a resulting minus No. 200 in the mixture in the middle of the range, about 4 percent?

- 5.26 Define the fineness modulus of aggregate. What is it used for?
- 5.27 Calculate the fineness modulus of aggregate B in problem 5.21. (Note that the percent passing the 1.18-mm (No. 16) sieve is not given and must be estimated.)
- 5.28 A portland cement concrete mix requires mixing sand having a gradation following the midpoint of the ASTM gradation band (Table 5.4) and gravel having a gradation following the midpoint of size number 467 of the ASTM gradation band (Table 5.5) at a ratio of 2:3 by weight. On a 0.45 power gradation chart, plot the gradations of the sand, gravel, and the blend. Is the gradation of the blend well graded? If not, what would you call it?
- 5.29 Discuss the effect of the amount of material passing the 0.075-mm (No. 200) sieve on the stability, drainage, and frost susceptibility of aggregate base courses.
- 5.30 Aggregates from three sources having the properties shown in Table P5.30 were blended at a ratio of 60:30:10 by weight. Determine the properties of the aggregate blend.

Table P5.30

Property	Aggregate 1	Aggregate 2	Sand
Coarse aggregate angularity, percent crushed faces	100	87	N/A
Bulk specific gravity	2.631	2.711	2.614
Apparent specific gravity	2.732	2.765	2.712

5.31 Three aggregates are blended by weight in the following percentages:

50% Crushed limestone	Bulk dry Sp. Gr. = 2.702
30% Blast furnace slag	Bulk dry Sp. Gr. = 2.331
20% Natural sand	Bulk dry Sp. Gr. = 2.609

What is the bulk specific gravity of the blended aggregates?

5.32 What is alkali–silica reactivity? What kind of problems are caused by ASR? Mention two ways to minimize ASR.

5.33 What are the typical deleterious substances in aggregates that affect portland cement concrete? Discuss these effects.

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6

PORTLAND CEMENT

Portland cement concrete is the most widely used manufactured construction material in the world. The importance of concrete in our daily lives cannot be overstated. It is used in structures such as buildings, bridges, tunnels, dams, factories, pavements, and playgrounds. Portland cement concrete consists of portland cement, aggregates, water, air voids, and, in many cases, admixtures. This chapter covers the topics of portland cement, mixing water, and admixtures; Chapter 7 will describe portland cement concrete.

There are many types of concrete, based on different cements. However, portland cement concrete is so prevalent that, unless otherwise identified, the term concrete is always assumed to mean portland cement concrete. Portland cement was patented by Joseph Aspdin in 1824 and was named after the limestone cliffs on the Isle of Portland in England (Kosmatka et al. 2002).

Portland cement is an instant glue (just add water) that bonds aggregates together to make portland cement concrete. Materials specialists concerned with the selection, specification, and quality control of civil engineering projects should understand the production, chemical composition, hydration rates, and physical properties of portland cement.

6.1 Portland Cement Production

Production of portland cement starts with two basic raw ingredients: a calcareous material and an argillaceous material. The calcareous material is a calcium oxide, such as limestone, chalk, or oyster shells. The argillaceous material is a combination of silica and alumina that can be obtained from clay, shale, and blast furnace slag. As shown in Figure 6.1, these materials are crushed and then stored in silos. The raw materials, in the desired proportions, are passed through a grinding mill, using either a wet or dry process. The ground material is stored until it can be sent to the kiln. Modern dry process cement plants use a heat recovery cycle to preheat the ground material, or feed

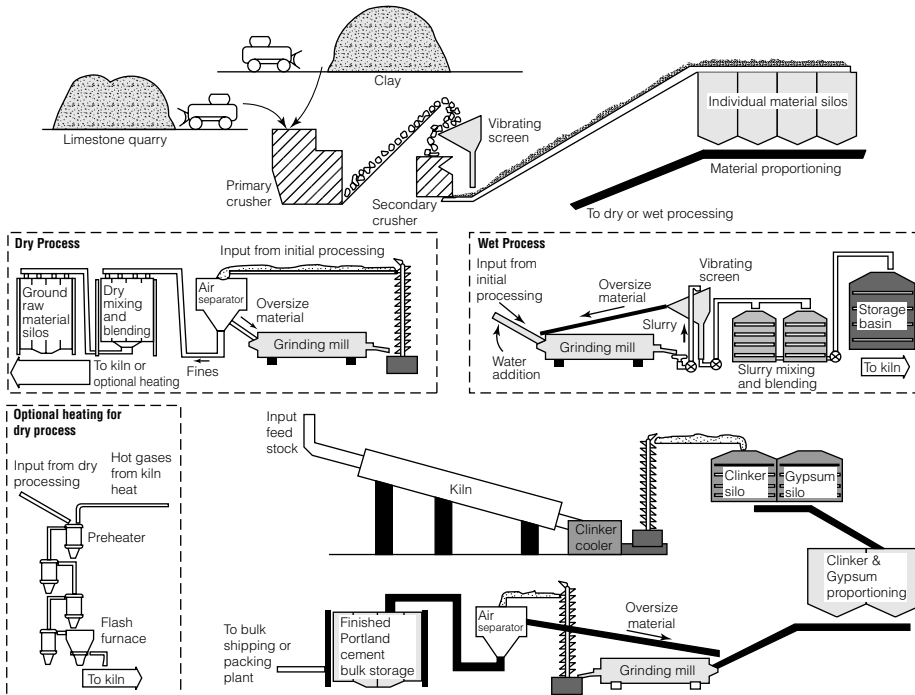


FIGURE 6.1 Steps in the manufacture of portland cement.

stock, with the exhaust gas from the kiln. In addition, some plants use a flash furnace to further heat the feed stock. Both the preheater and flash furnace improve the energy efficiency of cement production. In the kiln, the raw materials are melted at temperatures of 1400°C to 1650°C (2500°F to 3000°F), changing the raw materials into cement *clinker*. The clinker is cooled and stored. The final process involves grinding the clinker into a fine powder. During grinding, a small amount of gypsum is added to regulate the setting time of the cement in the concrete.

The finished product may be stored and transported in either bulk or sacks. In the United States, a standard *sack* of cement is 94 lb, which is approximately equal to 1 ft^3 of loose cement when freshly packed. The cement can be stored for long periods of time, provided it is kept dry.

6.2 Chemical Composition of Portland Cement

The raw materials used to manufacture portland cement are lime, silica, alumina, and iron oxide. These raw materials interact in the kiln, forming

TABLE 6.1 Main Compounds of Portland Cement.

Compound	Chemical Formula	Common Formula*	Usual Range by Weight (%)
Tricalcium Silicate	$3 \text{ CaO} \cdot \text{SiO}_2$	C_3S	45–60
Dicalcium Silicate	$2 \text{ CaO} \cdot \text{SiO}_2$	C_2S	15–30
Tricalcium Aluminate	$3 \text{ CaO} \cdot \text{Al}_2\text{O}_3$	C_3A	6–12
Tetracalcium Aluminoferrite	$4 \text{ CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Fe}_2\text{O}_3$	C_4AF	6–8

* The cement industry commonly uses shorthand notation for chemical formulas: C = Calcium oxide, S = silicon dioxide, A = Aluminum oxide, and F = Iron oxide.

complex chemical compounds. *Calcination* in the kiln restructures the molecular composition, producing four main compounds, as shown in Table 6.1.

C_3S and C_2S , when hydrated, provide the desired characteristics of the concrete. Alumina and iron, which produce C_3A and C_4AF , are included with the other raw materials to reduce the temperature required to produce C_3S from 2000°C to 1350°C (3500°F to 2500°F). This saves energy and reduces the cost of producing the portland cement.

In addition to these main compounds, there are minor compounds, such as magnesium oxide, titanium oxide, manganese oxide, sodium oxide, and potassium oxide. These minor compounds represent a few percent by weight of cement. The term minor compounds refers to their quantity and not to their importance. In fact, two of the minor compounds, sodium oxide (Na_2O) and potassium oxide (K_2O), are known as alkalis. These alkalis react with some aggregates causing the disintegration of concrete and affecting the rate of strength development, as discussed in Chapter 5.

6.3 Fineness of Portland Cement

Fineness of cement particles is an important property that must be carefully controlled. Since hydration starts at the surface of cement particles, the finer the cement particles, the larger the surface area and the faster the hydration. Therefore, finer material results in faster strength development and a greater initial heat of hydration. Increasing fineness beyond the requirements for a type of cement increases production costs and can be detrimental to the quality of the concrete.

The maximum size of the cement particles is 0.09 mm (0.0035 in.); 85% to 95% of the particles are smaller than 0.045 mm (0.0018 in.), and the average diameter is 0.01 mm (0.0004 in.). (For reference, a number 200 sieve



FIGURE 6.2 Blaine air permeability apparatus.

passes material smaller than 0.075 mm.) A kilogram of portland cement has approximately 7 trillion particles with a total surface area of about 300 m² to 400 m² (1500 ft² to 2000 ft² per pound). The total surface area per unit weight is a function of the size of the particles and is more readily measured. Thus, particle size specifications are defined in terms of the surface area per unit weight.

Fineness of cement is usually measured indirectly by measuring the surface area with the Blaine air permeability apparatus (ASTM C204) or the Wagner turbidimeter apparatus (ASTM C115). In the Blaine test (Figure 6.2), the surface area of the cement particles in cm²/g is determined by measuring the air permeability of a cement sample and relating it to the air permeability of a standard material. The Wagner turbidimeter determines the surface area by measuring the rate of sedimentation of cement suspended in kerosene. The finer the cement particles, the slower the sedimentation. Both the Blaine and Wagner tests are indirect measurements of surface area and use somewhat different measurement principles. Therefore, tests on a single sample of cement will produce different results. Fineness can also be measured by determining the percent passing the 0.045 mm sieve (No. 325) (ASTM C430).

6.4 Specific Gravity of Portland Cement

The specific gravity of cement is needed for mixture proportioning calculations. The specific gravity of portland cement (without voids between particles) is about 3.15 and can be determined according to ASTM C188. The density of the bulk cement (including voids between particles) varies considerably, depending on how it is handled and stored. For example, vibration during transportation of bulk cement consolidates the cement and increases its bulk density. Thus, cement quantities are specified and measured by weight rather than volume.

6.5 Hydration of Portland Cement

Hydration is the chemical reaction between the cement particles and water. The features of this reaction are the change in matter, the change in energy level, and the rate of reaction. The primary chemical reactions are shown in Table 6.2. Since portland cement is composed of several compounds, many reactions are occurring concurrently.

The hydration process occurs through two mechanisms: through-solution and topochemical. The through-solution process involves the following steps (Mehta and Monteiro 1993):

1. dissolution of anhydrous compounds into constituents
2. formation of hydrates in solution
3. precipitation of hydrates from the supersaturated solution

The through-solution mechanism dominates the early stages of hydration. Topochemical hydration is a solid-state chemical reaction occurring at the surface of the cement particles.

The aluminates hydrate much faster than the silicates. The reaction of tricalcium aluminate with water is immediate and liberates large amounts of heat. Gypsum is used to slow down the rate of aluminate hydration. The gypsum goes into the solution quickly, producing sulfate ions that suppress the solubility of the aluminates. The balance of aluminate to sulfate determines the rate of *setting* (solidification). Cement paste that sets at a normal rate requires low concentrations of both aluminate and sulfate ions. The cement paste will remain workable for about 45 minutes; thereafter, the paste starts to stiffen as crystals displace the water in the pores. The paste begins to solidify within 2 to 4 hours after the water is added to the cement. If there is an excess of both aluminate and sulfate ions, the workability stage may only last for 10 minutes and setting may occur in 1 to 2 hours. If the availability of aluminate ions is high, and sulfates are low, either a *quick set* (10 to 45 minutes) or *flash set* (less than 10 minutes) can occur. Finally, if the aluminate ions availability is low and the sulfate ions availability is high, the gypsum can recrystallize in the pores within 10 minutes, producing a flash set. Flash set is associated with large heat evolution and poor ultimate strength (Mehta and Monteiro 1993).

Calcium silicates combine with water to form calcium-silicate-hydrate, C-S-H. The crystals begin to form a few hours after the water and cement are mixed and can be developed continuously as long as there are unreacted cement particles and free water. C-S-H is not a well-defined compound. The calcium-to-silicate ratio varies between 1.5 and 2.0, and the structurally combined water content is more variable.

As shown in Table 6.2, the silicate hydration produces both C-S-H and calcium hydroxide. Complete hydration of C_3S produces 61% C-S-H and 39% calcium hydroxide; hydration of C_2S results in 82% C-S-H and 18% calcium hydroxide. Since C-S-H is what makes the hydrated cement paste strong, the

TABLE 6 . 2 Primary Chemical Reactions During Cement Hydration

$2(3 \text{ CaO} \cdot \text{SiO}_2)$ Tricalcium silicate	+ 6 H ₂ O Water	=	$3 \text{ CaO} \cdot 2 \text{ SiO}_2 \cdot 3 \text{ H}_2\text{O}$ + 3 Ca(OH) ₂ Calcium silicate hydrates + Calcium hydroxide
$2(2 \text{ CaO} \cdot \text{SiO}_2)$ Dicalcium silicate	+ 4 H ₂ O Water	=	$3 \text{ CaO} \cdot 2 \text{ SiO}_2 \cdot 3 \text{ H}_2\text{O}$ + Ca(OH) ₂ Calcium silicate hydrates + Calcium hydroxide
$3 \text{ CaO} \cdot \text{Al}_2\text{O}_3$ Tricalcium aluminate	+ 12 H ₂ O Water	=	$3 \text{ CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Ca(OH)}_2 \cdot 12 \text{ H}_2\text{O}$ Calcium aluminate hydrate
$4 \text{ CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Fe}_2\text{O}_3$ Tetracalcium aluminoferrite	+ 10 H ₂ O Water	=	$6 \text{ CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Fe}_2\text{O}_3 \cdot 12 \text{ H}_2\text{O}$ Calcium aluminoferrite hydrate
$3 \text{ CaO} \cdot \text{Al}_2\text{O}_3$ Tricalcium aluminate	+ 10 H ₂ O Water	=	$3 \text{ CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{CaSO}_4 \cdot 12 \text{ H}_2\text{O}$ Gypsum + Calcium monosulfoaluminate hydrate

ultimate strength of the concrete is enhanced by increasing the content of C_2S relative to the amount of C_3S . Furthermore, calcium hydroxide is susceptible to attack by sulfate and acidic waters. Increasing the proportion of C_2S relative to C_3S reduces the quantity of calcium hydroxide and, therefore, improves the durability of the concrete.

C_3S hydrates more rapidly than C_2S , contributing to the final set time and early strength gain of the cement paste. The rate of hydration is accelerated by sulfate ions in solution. Thus, a secondary effect of the addition of gypsum to cement is to increase the rate of development of the C-S-H.

6.5.1 ■ Structure Development in Cement Paste

The sequential development of the structure in a cement paste is summarized in Figure 6.3. The process begins immediately after water is added to the cement [Figure 6.3(a)]. In less than 10 minutes, the water becomes highly alkaline. As the cement particles hydrate, the volume of the cement particle reduces, increasing the space between the particles. During the early stages of hydration, weak bonds can form, particularly from the hydrated C_3A [Figure 6.3(b)]. Further hydration stiffens the mix and begins locking the structure of the material in place [Figure 6.3(c)]. Final set occurs when the C-S-H phase has developed a rigid structure, all components of the paste lock into place and the spacing between grains increases as the grains are consumed by hydration [Figure 6.3(d)]. The cement paste continues hardening and gains strength as hydration continues [Figure 6.3(e)]. Hardening develops rapidly at early ages and continues, as long as unhydrated cement particles and free water exist. However, the rate of hardening decreases with time.

6.5.2 ■ Evaluation of Hydration Progress

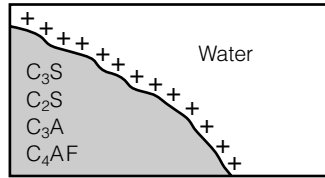
Several methods are available to evaluate the progress of cement hydration in hardened concrete. These include measuring the following properties (Neville 1981):

1. the heat of hydration
2. the amount of calcium hydroxide in the paste developed due to hydration
3. the specific gravity of the paste
4. the amount of chemically combined water
5. the amount of unhydrated cement paste using X-ray quantitative analysis
6. the strength of the hydrated paste, an indirect measurement

6.6 Voids in Hydrated Cement

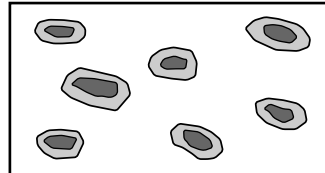
Due to the random growth of the crystals and the different types of crystals, voids are left in the paste structure as the cement hydrates. Concrete strength, durability, and volume stability are greatly influenced by voids.

The C-S-H phase is initially formed. C_3A forms a gel fastest.



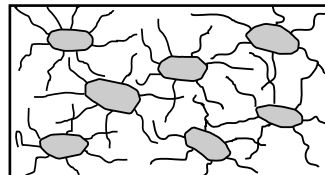
(a)

The volume of cement grain decreases as a gel forms at the surface. Cement grains are still able to move independently, but as hydration grows, weak interlocking begins. Part of the cement is in a thixotropic state; vibration can break the weak bonds.



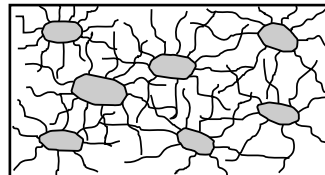
(b)

The initial set occurs with the development of a weak skeleton in which cement grains are held in place.



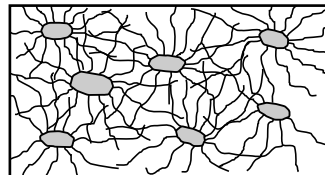
(c)

Final set occurs as the skeleton becomes rigid, cement particles are locked in place, and spacing between cement grains increases due to the volume reduction of the grains.



(d)

Spaces between the cement grains are filled with hydration products as cement paste develops strength and durability.



(e)

FIGURE 6.3 Development of structure in the cement paste: (a) initial C-S-H phase, (b) forming of gels, (c) initial set--development of weak skeleton, (d) final set--development of rigid skeleton, (e) hardening (Hover and Phillco 1990).

Two types of voids are formed during hydration: the interlayer hydration space and capillary voids.

Interlayer hydration space occurs between the layers in the C-S-H. The space thickness is between 0.5 nm and 2.5 nm, which is too small to affect the strength. It can, however, contribute 28% to the porosity of the paste. Water in the interparticle space is strongly held by hydrogen bonds, but

can be removed when humidity is less than 11%, resulting in considerable shrinkage.

Capillary voids are the result of the hydrated cement paste having a lower bulk specific gravity than the cement particles. The amount and size of capillary voids depends on the initial separation of the cement particles, which is largely controlled by the ratio of water to cement paste. For a highly hydrated cement paste in which a minimum amount of water was used, the capillary voids will be on the order of 10 nm to 50 nm. A poorly hydrated cement produced with excess water can have capillary voids on the order of 3 mm to 5 mm. Capillary voids greater than 50 nm decrease strength and increase permeability. Removal of water from capillary voids greater than 50 nm does not cause shrinkage, whereas removal of water from the smaller voids causes shrinkage.

In addition to the interlayer space and capillary voids, air can be trapped in the cement paste during mixing. The *trapped air* reduces strength and increases permeability. However, well-distributed, minute air bubbles can greatly increase the durability of the cement paste. Hence, as described later in this chapter, admixtures are widely used to *entrain air* into the cement paste.

6.7 Properties of Hydrated Cement

The proper hydration of portland cement is a fundamental quality control issue for cement producers. While specifications control the quality of the portland cement, they do not guarantee the quality of the concrete made with the cement. Mix design, quality control, and the characteristics of the mixing water and aggregates also influence the quality of the concrete. Properties of the hydrated cement are evaluated with either *cement paste* (water and cement) or *mortar* (paste and sand).

6.7.1 Setting

Setting refers to the stiffening of the cement paste or the change from a plastic state to a solid state. Although with setting comes some strength, it should be distinguished from *hardening*, which refers to the strength gain in a set cement paste. Setting is usually described by two levels: initial set and final set. The definitions of the initial and final sets are arbitrary, based on measurements by either the Vicat apparatus (ASTM C191) or the Gillmore needles (ASTM C266).

The Vicat test (Figure 6.4) requires that a sample of cement paste be prepared, using the amount of water required for normal consistency according to a specified procedure. The 1 mm (0.04 in.) diameter needle is allowed to penetrate the paste for 30 seconds and the amount of penetration is measured. The penetration process is repeated every 15 minutes (every 10 minutes for Type III cement) until a penetration of 25 mm (1 in.) or less is obtained. By interpolation, the time when a penetration of 25 mm occurs is

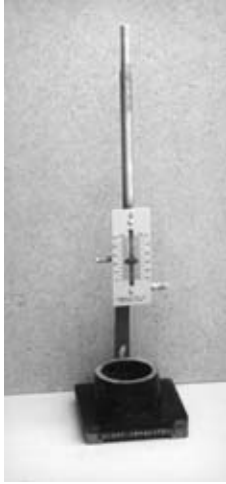


FIGURE 6.4 Vicat set time apparatus.

determined and recorded as the initial set time. The final set time is when the needle does not penetrate visibly into the paste.

Similar to the Vicat test, the Gillmore test (Figure 6.5) requires that a sample of cement paste of normal consistency be prepared. A pat with a flat top is molded and the initial Gillmore needle is applied lightly to its surface. The application process is repeated until the pat bears the force of the needle without appreciable indentation, and the elapsed time is recorded as the initial set time. This process is then repeated with the final Gillmore needle and the final set time is recorded. Due to the differences in the test apparatuses and procedures, the Vicat and Gillmore tests produce different results for a single sample of material.

The initial set time must allow for handling and placing the concrete before stiffening. The maximum final set time is specified and measured to ensure normal hydration. During cement manufacturing, gypsum is added to

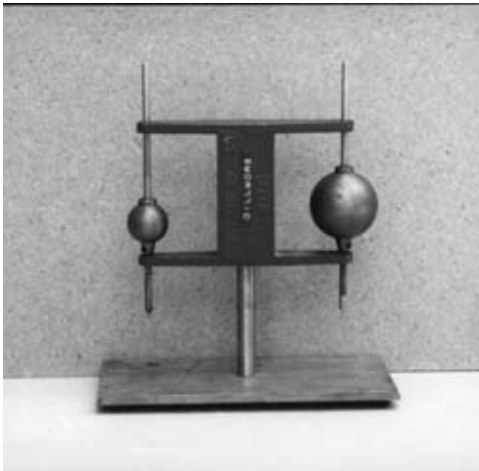


FIGURE 6.5 Gillmore set time apparatus.

regulate the setting time. Other factors that affect the set time include the fineness of the cement, the water–cement ratio, and the use of admixtures.

If the cement is exposed to humidity during storage, a *false set* might occur in which the cement stiffens within a few minutes of being mixed, without the evolution of much heat. To resolve this problem, the cement paste can be vigorously remixed, without adding water, in order to restore plasticity of the paste and to allow it to set in a normal manner without losing strength. A false set is different than a quick set and a flash set mentioned earlier; a false set can be remedied by remixing, whereas a quick set and a flash set cannot be remedied.

6.7.2 ■ Soundness

Soundness of the cement paste refers to its ability to retain its volume after setting. Expansion after setting, caused by delayed or slow hydration or other reactions, could result if the cement is unsound. The autoclave expansion test (Figure 6.6) (ASTM C151) is used to check the soundness of the cement paste. In this test, cement paste bars are subjected to heat and high pressure, and the amount of expansion is measured. ASTM C150 limits autoclave expansion to 0.8%.



FIGURE 6.6 Cement autoclave expansion apparatus.

6.7.3 ■ Compressive Strength

Compressive strength of mortar is measured by preparing 50-mm (2 in.) cubes and subjecting them to compression according to ASTM C109. The mortar is prepared with cement, water, and standard sand (ASTM C778). Minimum compressive strength values are specified by ASTM C150 for different cement types at different ages. The compressive strength of mortar cubes is proportional to the compressive strength of concrete cylinders. However, the compressive strength of the concrete cannot be predicted accurately from mortar cube strength, since the concrete strength is also affected by the aggregate characteristics, the concrete mixing, and the construction procedures.

6.8 Water–Cementitious Materials Ratio

In 1918, Abrams found that the ratio of the weight of water to the weight of cement, *water–cement ratio*, influences all the desirable qualities of concrete. For fully compacted concrete made with sound and clean aggregates, strength and other desirable properties are improved by reducing the weight of water used per unit weight of cement. This concept is frequently identified as Abrams's law.

Supplementary cementitious materials, such as fly ash, slag, silica fume, and natural pozzolans, have been used as admixtures in recent years to alter some of the properties of portland cement concrete. Therefore, the term water–cement ratio has been expanded to *water–cementitious materials ratio* to include these cementitious materials.

Hydration requires approximately 0.22 kg to 0.25 kg of water per 1 kg of cement. Concrete mixes generally require excess moisture, beyond the hydration needs, for workability. Excess water causes the development of capillary voids in the concrete. These voids increase the porosity and permeability of the concrete and reduce strength. Figure 6.7 shows a typical relationship between the water–cementitious materials ratio and compressive strength. It is easy to see that increasing the water–cementitious materials ratio decreases the compressive strength of the concrete for various curing times. A low water–cementitious materials ratio also increases resistance to weathering, provides a good bond between successive concrete layers, provides a good bond between concrete and steel reinforcement, and limits volume change due to wetting and drying. Air-entrained concrete includes an air entraining agent, an admixture, which is used to increase the concrete's resistance to freezing and thawing, as will be discussed later in this chapter. Curing maintains satisfactory moisture content and temperature in the hardened concrete for a definite period of time to allow for hydration (see Chapter 7).

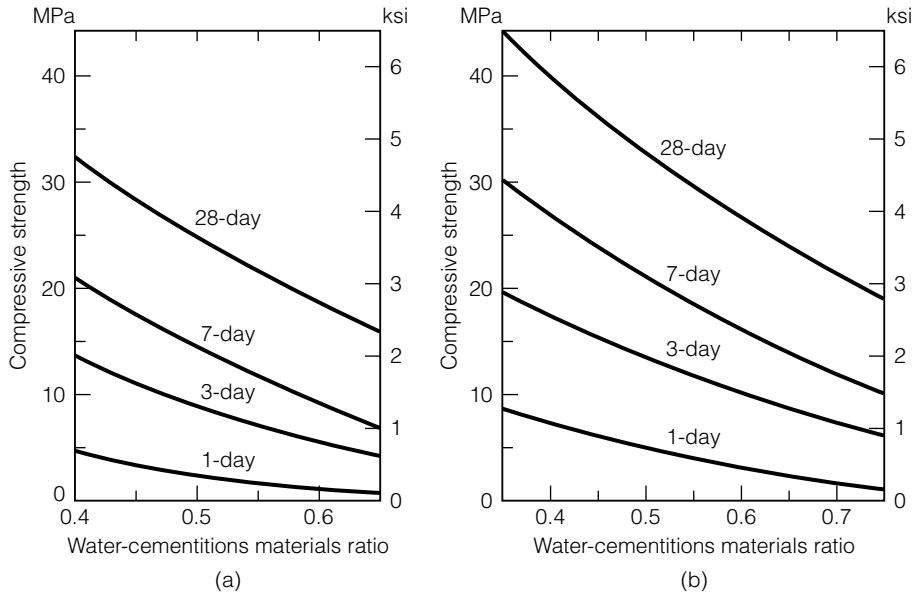


FIGURE 6.7 Typical age-strength relationships of concrete based on compression tests of 0.15 x 0.30 m (6 x 12 in.) cylinders, using Type I portland cement and moist-curing at 21C (70F): (a) air entrained concrete, (b) non-air entrained concrete (Kosmatka et al. 1988).

6.9 Types of Portland Cement

Different concrete applications require cements with different properties. Some applications require rapid strength gain to expedite the construction process. Other applications require a low heat of hydration to control volume change and associated shrinkage cracking. In some cases, the concrete is exposed to sulfates (SO_4), which can deteriorate normal portland cement concrete. Fortunately, these situations can be accommodated by varying the raw materials used to produce the cement, thereby altering the ratios of the four main compounds of portland cement listed in Table 6.1. The rate of hydration can also be altered by varying the fineness of the cement produced in the final grinding mill. Cement is classified to five standard types, as well as other special types.

6.9.1 Standard Portland Cement Types

Table 6.3 describes the five standard types of portland cement (Types I through V) specified by ASTM C150. In addition to these five types, air entrainers can be added to Type I, II, and III cements during manufacturing,

TABLE 6.3 Types and Applications of Standard Portland Cement

Type	Name	Application
I	Normal	General concrete work when the special properties of other types are not needed. Suitable for floors, reinforced concrete structures, pavements, etc.
II	Moderate Sulfate Resistance	Protection against moderate sulfate exposure, 0.1–0.2% weight water soluble sulfate in soil or 150–1500 ppm sulfate in water (sea water). Can be specified with a moderate heat of hydration, making it suitable for large piers, heavy abutments, and retaining walls. The moderate heat of hydration is also beneficial when placing concrete in warm weather.
III	High Early Strength	Used for fast-track construction when forms need to be removed as soon as possible or structure needs to be put in service as soon as possible. In cold weather, reduces time required for controlled curing.
IV	Low Heat of Hydration	Used when mass of structure, such as large dams, requires careful control of the heat of hydration.
V	High Sulfate Resistance	Protection from severe sulfate exposure, 0.2–2.0% weight water soluble sulfate in soils or 1500–10,800 ppm sulfate in water.

producing Types IA, IIA, and IIIA, which provide better resistance to freeze and thaw than do non-air-entrained cements. The use of air-entrained cements (Types IA, IIA, and IIIA) has diminished, due to improved availability and reliability of the air entrainer admixtures that can be added during concrete mixing. The uses and effects of air entrainers will be described in the section on admixtures. The ASTM specifications of the standard cement types are shown in Table 6.4.

The allowable maximum compound compositions are given in Table 6.5, along with the required Blaine fineness (controls particle size). Note that the chemical compositions of Type I and III cements are almost identical; the primary difference is the much greater surface area of the Type III cement. The C_3A contents of Type II and V cements are lower than that of Type I to improve sulfate resistance. C_3S and C_3A are limited in Type IV cement to limit the rate of hydration.

The existence of an ASTM specification for a type of cement does not guarantee that cement's availability. Type I cement is widely available and represents most of the United States' cement production. Type II is the second most available type. Cements can be manufactured that meet all the requirements of both Types I and II; these are labeled Type I/II. Type III cement represents about 4% of U.S. production. Due to the stricter grinding requirements for Type III, it is more expensive than Type I. The strength

TABLE 6.4 Standard Properties of Portland Cement (ASTM C150) (Copyright ASTM, reprinted with permission)

Cement Type	I	IA	II	IIA	III	IIIA	IV	V
Air content of mortar ^A , volume %								
max.	12	22	12	22	12	22	12	12
min.	—	16	—	16	—	16	—	—
Fineness ^B , specific surface, m ² /kg (alternative methods)								
Turbidimeter test, min.	160	160	160	160	—	—	160	160
Air permeability test, min.	280	280	280	280	—	—	280	280
Autoclave expansion, max., %	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80
Minimum compressive strength, psi (MPa) ^C								
1 day	—	—	—	—	1800	1450	—	—
	—	—	—	—	(12.4)	(10.0)	—	—
3 days	1800	1450	1500	1200	3500	2800	—	1200
	(12.4)	(15.5)	(10.3)	(8.3)	(24.1)	(19.3)	—	(8.3)
			1000 ^b	800 ^b				
			(6.9) ^D	(5.5) ^D				
7 days	2800	2250	2500	2000	—	—	1000	2200
	(19.3)	(15.5)	(17.2)	(13.8)	—	—	(6.9)	(15.2)
			1700 ^b	1350 ^D				
			(11.7) ^D	(9.3) ^D				
28 days	—	—	—	—	—	—	2500	3000
	—	—	—	—	—	—	(17.2)	(20.7)
Time of setting (alternative methods): ^E								
Gillmore test:								
Initial set, minutes, not less than	60	60	60	60	60	60	60	60
Final set, minutes, not more than	600	600	600	600	600	600	600	600
Vicat test:								
Initial set, minutes, not less than	45	45	45	45	45	45	45	45
Initial set, minutes, not more than	375	375	375	375	375	375	375	375

^A Compliance with the requirements of this specification does not necessarily ensure that the desired air content will be obtained in concrete.

^B Either of the two alternative fineness methods may be used at the option of the testing laboratory. However, when the sample fails to meet the requirements of the air-permeability test, the turbidimeter test shall be used and the requirements in this table for the turbidimeter method shall govern.

^C The strength at any specified test age shall be not less than that attained at any previous specified test age.

^D When the optional heat of hydration or the chemical limit on the sum of the tricalcium silicate and tricalcium aluminate is specified.

^E The purchaser should specify the type of setting-time test required. In case he does not so specify, the requirements of the Vicat test only shall govern.

TABLE 6.5 ASTM Chemical and Fineness Requirements for Portland Cement (ASTM C150)

Portland Cement Type	Maximum Compound Composition (%)				Blaine Fineness (m ² /kg)
	C ₃ S	C ₂ S	C ₃ A	C ₄ AF	
I	55	19	10	7	370
II	51	24	6	11	370
III	56	19	10	7	540
IV	28	49	4	12	380
V	38	43	4	9	380

gain of Type I cement can be accelerated by increasing the cement content per unit volume of concrete, so the selection of Type III becomes a question of economics and availability. Type IV can be manufactured on demand. As discussed later, adding fly ash to Type I or II portland cement reduces the heat of hydration, producing the benefits of Type IV, but at a lower cost. Type V cement is produced only in locations with a severe sulfate problem.

6.9.2 ■ Other Cement Types

Other than the five standard types of portland cement, several hydraulic cements are manufactured in the United States, including

- white portland cement
- blended hydraulic cements
 - portland blast furnace slag cement (Type IS)
 - portland-pozzolan cement (Type IP and Type P)
 - slag cement (Type S)
 - pozzolan-modified portland cement (Type I(PM))
 - slag-modified portland cement (Type I(SM))
- masonry cements
- expansive cements (Type K)
- specialty cements

In general, these cements have limited applications. Civil and construction engineers should be aware of their existence, but should study them further before using them.

6.10 Mixing Water

Any potable water is suitable for making concrete. However, some non-potable water may also be suitable. Frequently, material suppliers will use

unprocessed surface or well water if it can be obtained at a lower cost than processed water. However, impurities in the mixing water can affect concrete set time, strength, and long-term durability. In addition, chloride ions in the mixing water can accelerate corrosion of reinforcing steel.

6.10.1 ■ Acceptable Criteria

The acceptance criteria for questionable water are specified in ASTM C94. After 7 days, the compressive strength of mortar cubes made with the questionable water should not be less than 90% of the strength of cubes made with potable or distilled water (ASTM C109). Also, the set time of cement paste made with the questionable water should, as measured using the Vicat apparatus (ASTM C191), not be 1 hour less than or 1-1/2 hours more than the set time of paste made with potable or distilled water.

Other adverse effects caused by excessive impurities in mixing water include efflorescence (white stains forming on the concrete surface due to the formation of calcium carbonate), staining, corrosion of reinforcing steel, volume instability, and reduced durability. Therefore, in addition to the compressive strength and set time, there are maximum chemical limits that should not be exceeded in the mixing water, as shown in Table 6.6. Several tests are available to evaluate the chemical impurities of questionable water. Over 100 different compounds and ions can exist in the mixing water and can affect concrete quality; the more important effects are described in the Table 6.7.

6.10.2 ■ Disposal and Reuse of Concrete Wash Water

Disposal of waste water from ready-mixed concrete operations is a great concern of the ready-mixed concrete producers (Chini and Mbwambo 1996).

TABLE 6.6 Chemical Limits for Wash Water Used as Mixing Water (ASTM C94)
(Copyright ASTM, reprinted with permission)

Chemical	Maximum Concentration (ppm)	Test Method
Chloride, as Cl		ASTM D512
Prestressed concrete or concrete in bridge decks	500	
Other reinforced concrete in moist environments or containing aluminum embedments or dissimilar metals or with stay-in-place galvanized metal forms	1000	
Sulfate, as SO ₄	3000	ASTM D516
Alkalis, as (Na ₂ O + 0.658 K ₂ O)	600	
Total Solids	50,000	AASHTO T26

TABLE 6.7 Summary of Effects of Water Impurities on Concrete Quality

Impurity	Effect
Alkali Carbonate and Bicarbonate	Can retard or accelerate strength test setting and 28-day strength when total dissolved salts exceed 1000 ppm. Can also aggravate alkali–aggregate reaction.
Chloride	Corrosion of reinforcing steel is primary concern. Chloride can enter the mix through admixtures, aggregates, cement, and mixing water, so limits are expressed in terms of total free chloride ions. ACl limits water-soluble ion content based on the type of reinforcement: Prestressed concrete 0.06% Reinforced concrete exposed to chloride in service 0.15% Reinforced concrete protected from moisture 1.00% Other reinforced concrete 0.30%
Sulfate	Can cause expansive reaction and deterioration
Other Salts	Not harmful when concentrations limited to Calcium Bicarbonate 400 ppm Magnesium Bicarbonate 400 ppm Magnesium Sulfate 25,000 ppm Magnesium Chloride 40,000 ppm Iron Salts 40,000 ppm Sodium Sulfide 100 ppm
Sea Water	Do not use for reinforced concrete. Can accelerate strength gain but reduces ultimate strength. Can aggravate alkali reactions.
Acid Water	Limit concentrations of hydrochloric, sulfuric, and other inorganic acids to less than 10,000 ppm.
Alkaline Water	Possible increase in alkali–aggregate reactivity. Sodium hydroxide may introduce quick set at concentrations higher than 0.5%. Strength may be lowered. Potassium hydroxide in concentrations over 1.2% may reduce 28-day strength of some cements.
Sanitary Sewage Water	Dilute to reduce organic matter to less than 20 ppm.
Sugar	Concentrations over 500 ppm can retard setting time and alter strength development. Sucrose in the range of 0.03 to approximately 0.15% usually retards setting. Concentrations over 0.25% by weight of cement can accelerate strength gain, but substantially reduce 28-day strength.
Oils	Mineral oil (petroleum) in excess of 2.5% by weight of mix may reduce strength by 20%.
Algae	Can reduce hydration and entrain air. Do not use water containing algae.

This waste water is usually generated from truck wash systems, washing of central mixing plant, storm water runoff from the ready-mix plant yard, waste water generated from water sprayed dust control, and conveyor wash down. According to the Water Quality Act (part 116), waste water from

ready-mixed concrete operations is a hazardous substance (it contains caustic soda and potash) and its disposal is regulated by the Environmental Protection Agency (EPA). In addition, the high pH makes concrete wash water hazardous under the EPA definition of corrosivity.

The conventional practices for disposing of concrete wash water include dumping at the job site, dumping at a landfill, or dumping into a concrete wash water pit at the ready-mix plant. The availability of landfill sites for the disposal of concrete wash water has been drastically reduced in the past few decades. In addition, the current environmental restrictions either prevent or limit these conventional disposal practices. As a result, most ready-mix batch plants have developed a variety of operational configurations to manage their own wash water. The alternatives include settling ponds, storm water detention/retention facilities, and water reuse systems. Chemical stabilizing admixture systems have been used to circumvent the necessity to remove any wash water from concrete truck drums, and allow wash water to be reused for mixing more concrete. Studies have concluded that concrete properties are not significantly affected by the use of stabilized wash water (Borger et al. 1994).

Currently, concrete wash water is being used as mixing water for concrete in many places throughout the U.S. Some agencies, however, do not allow its use due to the existence of other impurities derived from concrete admixtures.

6.11 Admixtures for Concrete

Admixtures are ingredients other than portland cement, water, and aggregates that may be added to concrete to impart a specific quality to either the plastic (fresh) mix or the hardened concrete (ASTM C494). Some admixtures are charged into the mix as solutions. In such cases the liquid should be considered part of the mixing water. If admixtures cannot be added in solution, they are either weighed or measured by volume as recommended by the manufacturer. Admixtures are classified by the following chemical and functional physical characteristics (Hewlett 1978):

1. air entrainers
2. water reducers
3. retarders
4. hydration controller admixtures
5. accelerators
6. supplementary cementitious admixtures
7. specialty admixtures

The Portland Cement Association (PCA) identifies four major reasons for using admixtures (Kosmatka et al. 2002):

1. to reduce the cost of concrete construction
2. to achieve certain properties in concrete more effectively than by other means

3. to ensure quality of concrete during the stages of mixing, transporting, placing, and curing in adverse weather conditions
4. to overcome certain emergencies during concrete operations

6.11.1 ■ Air Entrainers

Air entrainers produce tiny air bubbles in the hardened concrete to provide space for water to expand upon freezing. As moisture within the concrete pore structure freezes, three mechanisms contribute to the development of internal stresses in the concrete:

1. Critical saturation—Upon freezing, water expands in volume by 9%. If the percent saturation exceeds 91.7%, the volume increase generates stress in the concrete.
2. Hydraulic pressure—Freezing water draws unfrozen water to it. The unfrozen water moving throughout the concrete pores generates stress, depending on length of flow path, rate of freezing, permeability, and concentration of salt in pores.
3. Osmotic pressure—Water moves from the gel to capillaries to satisfy thermodynamic equilibrium and to equalize alkali concentrations. Voids permit water to flow from the interlayer hydration space and capillaries into the air voids, where it has room to freeze without damaging the parts.

Internal stresses reduce the durability of hardened concrete, especially when cycles of freeze and thaw are repeated many times. The impact of each of these mechanisms is mitigated by providing a network of tiny air voids in the hardened concrete using air entrainers. In the late 1930s, the introduction of air entrainment in concrete represented a major advance in concrete technology. Air entrainment is recommended for all concrete exposed to freezing.

All concrete contains entrapped air voids, which have diameters of 1 mm or larger and which represent approximately 0.2% to 3% of the concrete volume. Entrained air voids have diameters that range from 0.01 mm to 1 mm, with the majority being less than 0.1 mm. The entrained air voids are not connected and have a total volume between 1% and 7.5% of the concrete volume. Concrete mixed for severe frost conditions should contain approximately 14 billion bubbles per cubic meter. Frost resistance improves with decreasing void size, and small voids reduce strength less than large ones. The fineness of air voids is measured by the specific surface index, equal to the total surface area of voids in a unit volume of paste. The specific surface index should exceed $23,600 \text{ m}^2/\text{m}^3$ ($600 \text{ in.}^2/\text{in.}^3$) for frost resistance.

In addition to improving durability, air entrainment provides other important benefits to both freshly mixed and hardened concrete. Air entrainment improves concrete's resistance to several destructive factors, including freeze-thaw cycles, deicers and salts, sulfates, and alkali-silica reactivity. Air entrainment also increases the workability of fresh concrete. Air entrainment decreases the strength of concrete, as shown in Figure 6.7; however,

this effect can be reduced for moderate-strength concrete by lowering the water–cementitious materials ratio and increasing the cement factor. High strength is difficult to attain with air-entrained concrete.

Air-entraining admixtures are available from several manufacturers and can be composed of a variety of materials, such as

- salts of wood resins (Vinsol resin)
- synthetic detergents
- salts of sulfonated lignin (by-product of paper production)
- salts of petroleum acids
- salts of proteinaceous material
- fatty and resinous acids
- alkylbenzene sulfonates
- salts of sulfonated hydrocarbons

Air entrainers are usually liquid and should meet the specifications of ASTM C260. The agents enhance air entrainment by lowering the surface tension of the mixing water. Anionic air entrainers are hydrophobic (water hating). The negative charge of the agent is attracted to the positive charge of the cement particle. The hydrophobic agent forms tough, elastic, air-filled bubbles. Mixing disperses the air bubbles throughout the paste, and the sand particles form a grid that holds the air bubbles in place. Other types of air entrainers have different mechanisms but produce similar results.

6.11.2 ■ Water Reducers

Workability of fresh or plastic concrete requires more water than is needed for hydration. The excess water, beyond the hydration requirements, is detrimental to all desirable properties of hardened concrete. Therefore, water-reducing admixtures have been developed to gain workability and, at the same time, maintain quality. Water reducers increase the mobility of the cement particles in the plastic mix, allowing workability to be achieved at lower water contents. Water reducers are produced with different levels of effectiveness: conventional, mid-range, and high-range. Figure 6.8 shows concrete without the addition of admixture and with the addition of conventional, mid-range, and high-range water reducers. As shown in the figure, the slump of the concrete increases, indicating an increase in workability. The high-range water reducer is typically called superplasticizer.

Water Reducers Mechanism Cement grains develop static electric charge on their surface as a result of the cement-grinding process. Unlike charges attract, causing the cement grains to cluster or “floccluate” [Figure 6.9(a)], which in turn limits the workability. The chemicals in the water-reducing admixtures reduce the static attraction among cement particles. The molecules of water-reducing admixtures have both positive and negative charges at one end, and a single charge (usually negative) on the other end, as illustrated in Figure 6.9(b). These molecules are attracted by the charged surface of the cement grains. The water reducers neutralize the static attraction on

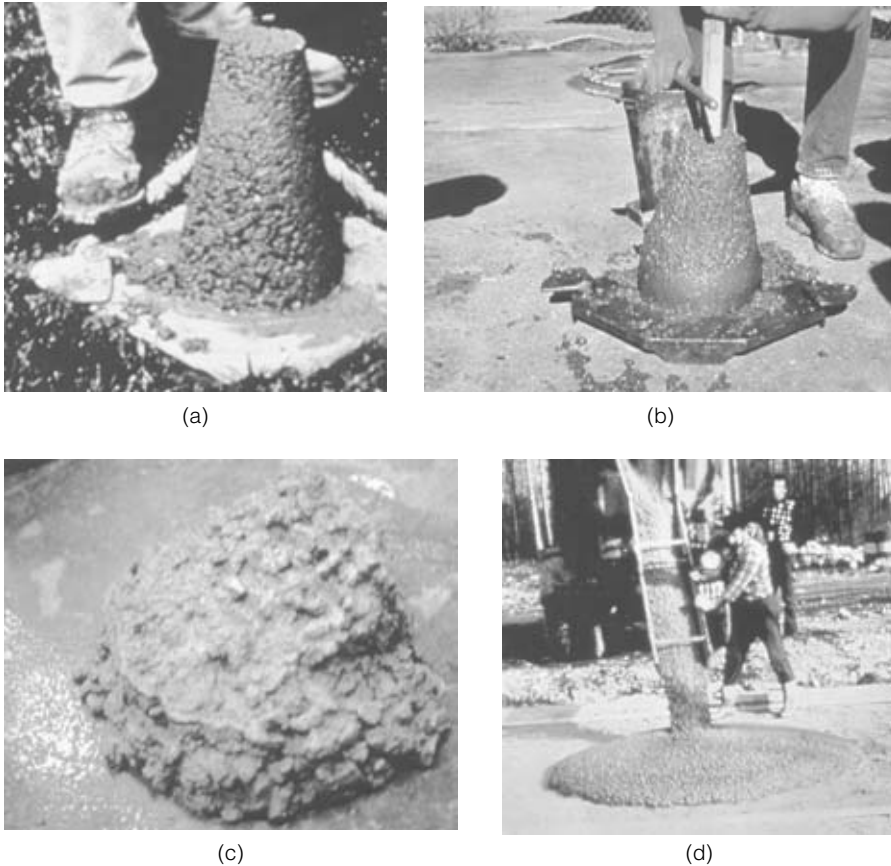


FIGURE 6.8 Slumps of concretes with the same water-cement ratio: (a) no water reducer, (b) conventional water reducer, (c) mid-range water reducer, (d) high-range water reducer (Superplasticizer).

the cement surfaces. As a result, the clusters of cement grains are broken apart. Mutual repulsion of like charges pushes the cement grains apart, achieving a better distribution of particles [see Figure 6.9(c)], more uniform hydration, and a less-viscous paste.

Water reducing admixtures can be used indirectly to gain strength. Since the water-reducing admixture increases workability, we can take advantage of this phenomenon to decrease the mixing water, which in turn reduces the water-cementitious materials ratio and increases strength. Hewlett (1978) demonstrated that water reducers can actually be used to accomplish three different objectives, as shown in Table 6.8.

1. Adding a water reducer without altering the other quantities in the mix increases the slump, which is a measure of concrete consistency and an indicator of workability, as will be discussed in Chapter 7.

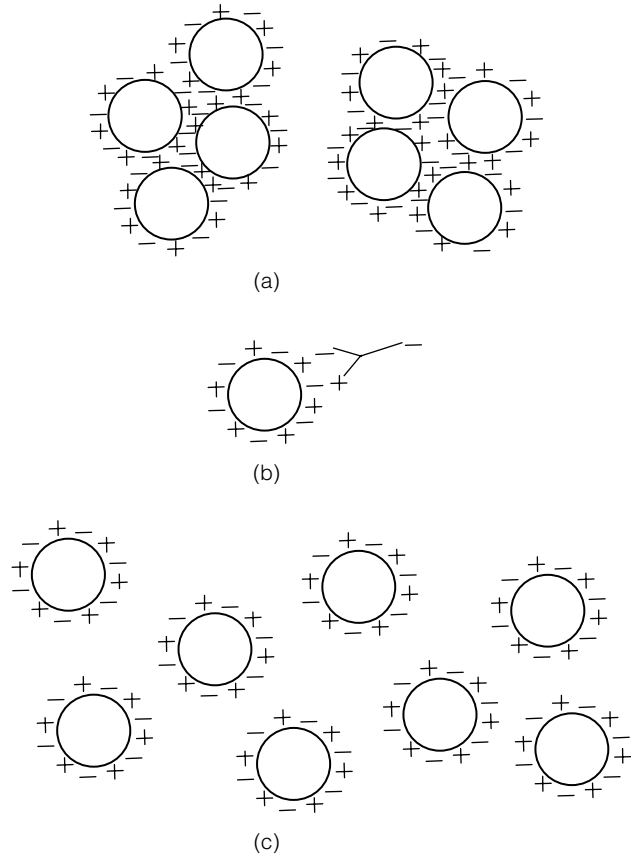


FIGURE 6.9 Water reducer mechanism: (a) clustering of cement grains without water reducer, (b) molecule of water reducer, and (c) better distribution of cement grains due to the use of water reducer.

TABLE 6.8 Effects of Water Reducer

	Cement Content kg/m ³	Water–Cement Ratio	Slump (mm)	Compressive Strength (MPa)	
				7 day	28 day
Base Mix	300	0.62	50	25	37
Improve Consistency	300	0.62	100	26	38
Increase Strength	300	0.56	50	34	46
Reduce Cost	270	0.62	50	25.5	37.5

2. The strength of the mix can be increased by using the water reducer by lowering the quantity of water and keeping the cement content constant.
3. The cost of the mix, which is primarily determined by the amount of cement, can be reduced. In this case, the water reducer allows a decrease in the amount of water. The amount of cement is then reduced to keep the water–cementitious materials ratio equal to the original mix. Thus, the quality of the mix, as measured by compressive strength, is kept constant, although the amount of cement is decreased.

Superplasticizers Superplasticizers, or high-range water reducers, can either greatly increase the flow of the fresh concrete or reduce the amount of water required for a given consistency. For example, adding a superplasticizer to a concrete with a 75-mm (3 in.) slump can increase the slump to 230 mm (9 in.), or the original slump can be maintained by reducing the water content 12% to 30%. Reducing the amount of mixing water reduces the water–cementitious materials ratio, which in turn, increases the strength of hardened concrete. In fact, the use of superplasticizers has resulted in a major breakthrough in the concrete industry. Now, high-strength concrete in the order of 70–80 MPa (10,000–12,000 psi) compressive strength or more can be produced when superplasticizers are used. Superplasticizers can be used in the following cases:

1. a low water–cementitious materials ratio is beneficial (e.g., high-strength concrete, early strength gain, and reduced porosity)
2. placing thin sections
3. placing concrete around tightly spaced reinforcing steel
4. placing cement underwater
5. placing concrete by pumping
6. consolidating the concrete is difficult

When superplasticizers are used, the fresh concrete stays workable for a short time, 30 min to 60 min, and is followed by rapid loss in workability. Superplasticizers are usually added at the plant to ensure consistency of the concrete. In critical situations, they can be added at the jobsite, but the concrete should be thoroughly mixed following the addition of the admixture. The setting time varies with the type of agents, the amount used, and the interactions with other admixtures used in the concrete.

6.11.3 ■ Retarders

Some construction conditions require that the time between mixing and placing or finishing the concrete be increased. In such cases, retarders can be used to delay the initial set of concrete. Retarders are used for several reasons, such as the following:

1. offsetting the effect of hot weather
2. allowing for unusual placement or long haul distances
3. providing time for special finishes (e.g., exposed aggregate)

Retarders can reduce the strength of concrete at early ages (e.g., one to three days). In addition, some retarders entrain air and improve workability. Other retarders increase the time required for the initial set but reduce the time between the initial and final set. The properties of retarders vary with the materials used in the mix and with job conditions. Thus, the use and effect of retarders must be evaluated experimentally during the mix design process.

6.11.4 ■ Hydration-Control Admixtures

These admixtures have the ability to stop and reactivate the hydration process of concrete. They consist of two parts: a stabilizer and an activator. Adding the stabilizer completely stops the hydration of the cementing materials for up to 72 hours, while adding the activator to the stabilized concrete reestablishes normal hydration and setting. These admixtures are very useful in extending the use of ready-mixed concrete when the work at the jobsite is stopped for various reasons. They are also useful when concrete is being hauled for a long time.

6.11.5 ■ Accelerators

Accelerators are used to develop early strength of concrete at a faster rate than that developed in normal concrete. The ultimate strength, however, of high early strength concrete is about the same as that of normal concrete. Accelerators are used to

1. reduce the amount of time before finishing operations begin
2. reduce curing time
3. increase rate of strength gain
4. plug leaks under hydraulic pressure efficiently

The first three reasons are particularly applicable to concrete work placed during cold temperatures. The increased strength gained helps to protect the concrete from freezing and the rapid rate of hydration generates heat that can reduce the risk of freezing.

Calcium chloride, CaCl_2 , is the most widely used accelerator (ASTM D98). Both initial and final set times are reduced with calcium chloride. The initial set time of 3 hours for a typical concrete can be reduced to 1.5 hours by adding an amount of calcium chloride equal to 1% of the cement weight; 2% reduces the initial set time to 1 hour. Typical final set times are 6 hours, 3 hours, and 2 hours for 0%, 1%, and 2% calcium chloride. Figure 6.10 shows that strength development is also affected by CaCl_2 for plain portland cement concrete (PCC) and portland cement concrete with 2% calcium chloride. Concrete with CaCl_2 develops higher early strength compared with plain concrete cured at the same temperature (Hewlett 1978).

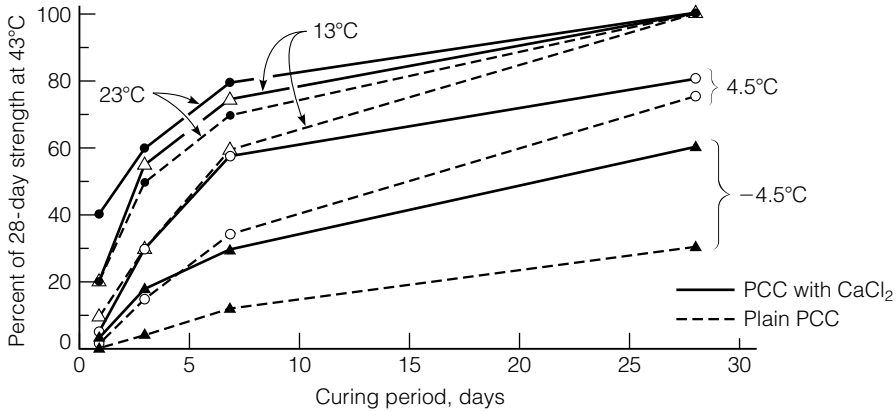


FIGURE 6.10 Effect of CaCl₂ on strength development at different curing temperatures.

The PCA recommends against using calcium chloride under the following conditions:

1. concrete is prestressed
2. concrete contains embedded aluminum such as conduits, especially if the aluminum is in contact with steel
3. concrete is subjected to alkali-aggregate reaction
4. concrete is in contact with water or soils containing sulfates
5. concrete is placed during hot weather
6. mass applications of concrete

The American Concrete Institute (ACI) recommends the following limits to water-soluble chloride ion content based on percent weight of cement (American Concrete Institute 1986):

Member Type	Chloride Ion Limit, %
Prestressed concrete	0.06
Reinforced concrete subjected to chloride in service	0.15
Reinforced concrete protected from moisture	1.00
Other reinforced concrete	0.30

Several alternatives to the use of calcium chloride are available. These include the following:

1. using high early strength (Type III) cement
2. increasing cement content

3. curing at higher temperatures
4. using non-calcium chloride accelerators such as triethanolamine, sodium thiocyanate, calcium formate, calcium nitrite, or calcium nitrate

6.11.6 ■ Supplementary Cementitious Admixtures

Several byproducts of other industries have been used in concrete as supplementary cementitious admixtures since the 1970s, especially in North America. These materials have been used to improve some properties of concrete and to reduce the problem of discarding them. Since these materials are cementitious, they can be used in addition to or as a partial replacement for portland cement. In fact, two or more of these supplementary cementitious additives have been used together to enhance concrete properties. These supplementary cementitious materials include fly ash, ground granulated blast furnace slag, silica fume, and natural pozzolans.

Fly Ash Fly ash is the most commonly used pozzolan in civil engineering structures. Fly ash is a by-product of the coal industry. Combusting pulverized coal in an electric power plant burns off the carbon and most volatile materials. However, depending on the source and type of coal, a significant amount of impurities passes through the combustion chamber. The carbon contents of common coals ranges from 70 to 100 percent. The noncarbon percentages are impurities (e.g., clay, feldspar, quartz, and shale), which fuse as they pass through the combustion chamber. Exhaust gas carries the fused material, fly ash, out of the combustion chamber. The fly ash cools into spheres, which may be solid, hollow (cenospheres), or hollow and filled with other spheres (plerospheres). Particle diameters range from 1 μm to more than 0.1 mm, with an average of 0.015 mm to 0.020 mm, and are 70% to 90% smaller than 0.045 mm. Fly ash is primarily a silica glass composed of silica (SiO_2), alumina (Al_2O_3), iron oxide (Fe_2O_3), and lime (CaO). Fly ash is classified (ASTM C618) as follows:

Class N—Raw or calcined natural pozzolans, including diatomaceous earths, opaline cherts and shales, ruffs and volcanic ashes or pumicites, and some calcined clays and shales

Class F—Fly ash with pozzolan properties

Class C—Fly ash with pozzolan and cementitious properties

Class F fly ash usually has less than 5% CaO but may contain up to 10%. Class C fly ash has 15% to 30% CaO.

The spherical shape of fly ash increases the workability of the fresh concrete. In addition, fly ash extends the hydration process, allowing a greater strength development and reduced porosity. Studies have shown that concrete containing more than 20% fly ash by weight of cement has a much smaller pore size distribution than portland cement concrete without fly ash. The lower heat of hydration reduces the early strength of the concrete. The extended reaction permits a continuous gaining of strength beyond what can be accomplished with plain portland cement.

Ground Granulated Blast Furnace Slag Ground granulated blast furnace slag (GGBF slag) is made from iron blast furnace slag. It is a nonmetallic hydraulic cement consisting basically of silicates and aluminosilicates of calcium, which is developed in a molten condition simultaneously with iron in a blast furnace. The molten slag is rapidly chilled by quenching in water to form a glassy sandlike granulated material. The material is then ground to less than 45 microns. The specific gravity of GGBF slag is in the range of 2.85 to 2.95.

The rough and angular-shaped ground slag in the presence of water and an activator, NaOH or CaOH, both supplied by portland cement, hydrates and sets in a manner similar to portland cement.

Ground slag has been used as a cementitious material in concrete since the beginning of the 1900s. Ground granulated blast furnace slag commonly constitutes between 30% and 45% of the cementing material in the mix. Some slag concretes have a slag component of 70% or more of the cementitious material. ASTM C 989 (AASHTO M 302) classifies slag by its increasing level of reactivity as Grade 80, 100, or 120.

Silica Fume Silica fume is a byproduct of the production of silicon metal or ferrosilicon alloys. One of the most beneficial uses for silica fume is as a mineral admixture in concrete. Because of its chemical and physical properties, it is a very reactive pozzolan. Concrete containing silica fume can have very high strength and can be very durable. Silica fume is available from suppliers of concrete admixtures and, when specified, is simply added during concrete production either in wet or dry forms. Placing, finishing, and curing silica fume concrete require special attention on the part of the concrete contractor.

Silicon metal and alloys are produced in electric furnaces. The raw materials are quartz, coal, and woodchips. The smoke that results from furnace operation is collected and sold as silica fume.

Silica fume consists primarily of amorphous (noncrystalline) silicon dioxide (SiO_2). The individual particles are extremely small, approximately 1/100th the size of an average cement particle. Because of its fine particles, large surface area, and the high SiO_2 content, silica fume is a very reactive pozzolan when used in concrete. The quality of silica fume is specified by ASTM C 1240 and AASHTO M 307.

In addition to producing high-strength concrete, silica fume can reduce concrete corrosion induced by deicing or marine salts. Silica fume concrete with a low water content is highly resistant to penetration by chloride ions. More information is available at the www.silicafume.org website.

Natural Pozzolans A pozzolan is a siliceous and aluminous material which, in itself, possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, react chemically with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (ASTM C595). Naturally occurring pozzolans, such as fine volcanic ash, combined with burned lime, were used about 2000 years ago for building construction, and pozzolan continues to be used today. As shown in Table 6.2, calcium hydroxide is one of the products generated by the hydration of C_3S and C_2S . In fact, up to 15% of the weight of portland

cement is hydrated lime. Adding a pozzolan to portland cement generates an opportunity to convert this free lime to a cementitious material.

Tables 6.9 and 6.10 summarize the effects of supplementary cementitious admixtures on fresh and hardened concrete. These summaries are based on general trends and should be verified experimentally for specific materials and construction conditions.

6.11.7 ■ Specialty Admixtures

In addition to the admixtures previously mentioned, several admixtures are available to improve concrete quality in particular ways. The civil engineer

TABLE 6.9 Effect of Supplementary Cementitious Admixtures on Freshly Mixed Concrete

Quality Measure	Effect
Water Requirements	Fly ash reduces water requirements. Silica fume increases water requirements.
Air Content	Fly ash and silica fume reduce air content, compensate by increasing air entrainer.
Workability	Fly ash, ground slag, and inert minerals generally increase workability. Silica fume reduces workability; use superplasticizer to compensate.
Hydration	Fly ash reduces heat of hydration. Silica fume may not affect heat, but superplasticizer used with silica fume can increase heat.
Set Time	Fly ash, natural pozzolans and blast furnace slag increase set time. Can use accelerator to compensate.

TABLE 6.10 Effect of Supplementary Cementitious Admixtures on Hardened Concrete

Quality Measure	Effect
Strength	Fly ash increases ultimate strength, but reduces rate of strength gain. Silica fume has less effect on rate of strength gain than pozzolans.
Drying shrinkage and creep	Low concentrations usually have little effect. High concentrations of ground slag or fly ash may increase shrinkage. Silica fume may reduce shrinkage.
Permeability and absorption	Generally reduce permeability and absorption. Silica fume is especially effective.
Alkali–aggregate reactivity	Generally reduced reactivity. Extent of improvement depends on type of admixture.
Sulfate resistance	Improved due to reduced permeability.

should be aware of these admixtures, but will need to study their application in detail, as well as their cost, before using them. Examples of specialty admixtures include

- workability agents
- corrosion inhibitors
- damp-proofing agents
- permeability-reducing agents
- fungicidal, germicidal, and insecticidal admixtures
- pumping aids
- bonding agents
- grouting agents
- gas-forming agents
- coloring agents
- shrinkage reducing

S U M M A R Y

The development of portland cement as the binder material for concrete is one of the most important innovations of civil engineering. It is extremely difficult to find civil engineering projects that do not include some component constructed with portland cement concrete. The properties of portland cement are governed by the chemical composition and the fineness of the particles. These control the rate of hydration and the ultimate strength of the concrete. Abrams's discovery of the importance of the water-to-cementitious materials ratio as the factor that controls the quality of concrete is perhaps the single most important advance in concrete technology. Second to this development was the introduction of air entrainment. The subsequent development of additional admixtures for concrete has improved the workability, set time, strength, and economy of concrete construction.

Q U E S T I O N S A N D P R O B L E M S

- 6.1 What ingredients are used for the production of portland cement?
- 6.2 What is the role of gypsum in the production of portland cement?
- 6.3 What is a typical value for the fineness of portland cement?

- 6.4 What are the primary chemical reactions during the hydration of portland cement?
- 6.5 Define the C-S-H phase of cement paste.
- 6.6 What are the four main chemical compounds in portland cement?
- 6.7 What chemical compounds contribute to early strength gain?
- 6.8 Define
- interlayer hydration space
 - capillary voids
 - entrained air
 - entrapped air
- 6.9 Define *initial set* and *final set*. Briefly discuss one method used to determine them.
- 6.10 The following laboratory tests are performed:
- Setting time test of cement paste samples
 - Compressive strength of mortar cubes
- What are the significance and use of each of these tests?
- 6.11 What is a false set of portland cement? State one reason for false set. If false set is encountered at the job site, what would you do?
- 6.12 You are an engineer in charge of mixing concrete in an undeveloped area where no potable water is available for mixing concrete. A source of water is available that has some impurities. What tests would you run to evaluate the suitability of this water for concrete mixing? What criteria would you use?
- 6.13 The water–cementitious materials ratio is important because it influences all of the desirable qualities of concrete.
- What is a typical water–cementitious materials ratio for normal strength concrete?
 - What is the minimum water–cementitious materials ratio for hydration only?
 - Why is the extra water necessary?
 - Briefly describe how super high strength concrete ($f'_c = 15,000$ psi) can be made.
- 6.14 Discuss the effect of water-to–cementitious materials ratio on the quality of hardened concrete. Explain why this effect happens.
- 6.15 Draw a graph to show the general relationship between the compressive strength of the concrete and the water-to–cementitious materials ratio for different curing times. Label all axes and curves.
- 6.16 What are the five primary types and functions of portland cement? Describe an application for each type.
- 6.17 Why isn't pozzolan used with Type III cement?
- 6.18 What type of cement would you use in each of the following cases? Why?
- Construction of a large pier
 - Construction in cold weather

- c. Construction in a warm climate region such as the Phoenix area
 - d. Concrete structure without any specific exposure condition
 - e. Building foundation in a soil with severe sulfate exposure
- 6.19 In order to evaluate the suitability of nonpotable water available at the job site for mixing concrete, six standard mortar cubes were made using that water and six others using potable water. The cubes were tested for compressive strength after 7 days of curing and produced the following loads to failure (in pounds).

Cubes Made with Nonpotable Water	Cubes Made with Potable Water
17,810	16,730
15,110	18,870
14,200	15,230
18,290	17,470
14,650	16,990
16,430	17,850

- a. Based on these results only, would you accept that water for mixing concrete according to ASTM C94?
 - b. According to ASTM C94, are there other tests to be performed to evaluate the suitability of that water? Discuss briefly.
- 6.20 Three standard mortar cubes were made using nonpotable water available at the job site, and three others were made using potable water. The cubes were tested for compressive strength after 7 days of curing and produced the following failure loads in kips:

Nonpotable water	Potable water
15.2	17.1
14.5	18.6
15.9	16.2

- Based on these results only, would you accept that water for mixing concrete according to ASTM standards? Explain why.
- 6.21 Discuss the problem of disposal of waste water from ready-mixed concrete operations. State three alternate methods that can be used to alleviate this problem.
 - 6.22 State five types of admixtures and discuss their applications.
 - 6.23 If you were a materials engineer in Minnesota (cold climate) and could use only one type of admixture, which would you select? Explain.

- 6.24 Under what condition is an air-entraining agent is needed? Why? Discuss how the air-entraining agent performs its function.
- 6.25 If a water reducer is added to the concrete mix without changing other ingredients, what will happen to the properties of the concrete? If the intention of adding the water reducer is to increase the compressive strength of hardened concrete, how can this be achieved?
- 6.26 Why is a superplasticizer used? How does it perform its function?
- 6.27 A concrete mix includes the following ingredients per cubic foot:

Cement = 25 lb
 Water = 11 lb
 No admixture

Table P6.27 shows possible changes that can be made to the mix ingredients. Indicate in the appropriate boxes in the table what will happen in each case for the workability and the ultimate compressive strength as *increase*, *decrease*, or *approximately the same*.

Table P6.27

Cement (lb)	Water (lb)	Admixture	What will happen?	
			Workability	Ultimate Compressive Strength
25	15	None		
28	11	None		
25	11	Water reducer		
25	8	Water reducer		
25	11	Superplasticizer		
25	11	Air entrainer		
25	11	Accelerator		

- 6.28 A materials engineer is working in a research project to evaluate the effect of one type of admixture on the compressive strength of concrete. She tested 10 mortar cubes made without admixture and 10 others with admixture after 28 days of curing. The compressive strengths of cubes without admixture were 25.1, 24.4, 25.8, 25.2, 23.9, 24.7, 24.3, 26.0, 23.8, and 24.6 MPa. The compressive strengths of cubes with admixture were 25.3, 26.8, 26.5, 24.5, 27.2, 24.8, 24.1, 25.9, 25.3, and 25.0 MPa. Using the statistical *t*-test, does this admixture show an increase of the compressive strength of the cement mortar at a level of significance of 0.05?
- 6.29 A materials engineer is working in a research project to evaluate the effect of one type of admixture on the compressive strength of concrete. He tested 8 mortar cubes made with admixture and 8 others without

admixture after 28 days of curing. The compressive strengths of cubes in psi with and without admixture are shown in Table P6.29.

Table P6.29

Cube No.	Compressive strength, psi	
	With admixture	Without admixture
1	3520	3550
2	3645	3785
3	3715	3720
4	3500	3645
5	3414	3560
6	3685	3645
7	3545	3400
8	3515	3555

Using the statistical t -test, is there a significant difference between the means of the compressive strengths of the two cement mortars at a level of significance of 0.10?

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7

PORTLAND CEMENT CONCRETE



Civil and construction engineers are directly responsible for the quality control of portland cement concrete and the proportions of the components used in it. The quality of the concrete is governed by the chemical composition of the portland cement, hydration and development of the microstructure, admixtures, and aggregate characteristics. The quality is strongly affected by placement, consolidation, and curing, as well.

How a concrete structure performs throughout its service life is largely affected by the methods of mixing, transporting, placing, and curing the concrete in the field. In fact, the ingredients of a “good” concrete may be the same as those of a “bad” concrete. The difference, however, is often the expertise of the engineer and technicians who are handling the concrete during construction.

Because of the advances made in concrete technology in the past few decades, concrete can be used in many more applications. Civil and construction engineers should be aware of the alternatives to conventional concrete, such as lightweight concrete, high-strength concrete, polymer concrete, fiber-reinforced concrete, and roller-compacted concrete. Before using these alternatives to conventional concrete, the engineer needs to study them, and their costs, in detail. This chapter covers basic principles of conventional portland cement concrete, its proportioning, mixing and handling, curing, and testing. Alternatives to conventional concrete that increase the applications and improve the performance of concrete are also introduced.

7.1 Proportioning of Concrete Mixes

The properties of concrete depend on the mix proportions and the placing and curing methods. Designers generally specify or assume a certain strength or modulus of elasticity of the concrete when determining structural dimensions.

The materials engineer is responsible for assuring that the concrete is properly proportioned, mixed, placed, and cured so to have the properties specified by the designer.

The proportioning of the concrete mix affects its properties in both the plastic and solid states. During the plastic state, the materials engineer is concerned with the workability and finishing characteristics of the concrete. Properties of the hardened concrete important to the materials engineer are the strength, modulus of elasticity, durability, and porosity. Strength is generally the controlling design factor. Unless otherwise specified, concrete strength f'_c refers to the average compressive strength of three tests. Each test is the average result of two 0.15-m \times 0.30-m (6-in. \times 12-in.) cylinders tested in compression after curing for 28 days.

The PCA specifies three qualities required of properly proportioned concrete mixtures (Kosmatka et al. 2002):

1. acceptable workability of freshly mixed concrete
2. durability, strength, and uniform appearance of hardened concrete
3. economy

In order to achieve these characteristics, the materials engineer must determine the proportions of cement, water, fine and coarse aggregates, and the use of admixtures. Several mix design methods have been developed over the years, ranging from an arbitrary volume method (1:2:3 cement: sand: coarse aggregate) to the weight and absolute volume methods prescribed by the American Concrete Institute's Committee 211. The weight method provides relatively simple techniques for estimating mix proportions, using an assumed or known unit weight of concrete. The absolute volume method uses the specific gravity of each ingredient to calculate the unit volume each will occupy in a unit volume of concrete. The absolute volume method is more accurate than the weight method. The mix design process for the weight and absolute volume methods differs only in how the amount of fine aggregates is determined.

7.1.1 ■ Basic Steps for Weight and Absolute Volume Methods

The basic steps required for determining mix design proportions for both weight and absolute volume methods are as follows (Kosmatka et al. 2002):

1. Evaluate strength requirements.
2. Determine the water–cementitious materials ratio required.
3. Evaluate coarse aggregate requirements.
 - maximum aggregate size of the coarse aggregate
 - quantity of the coarse aggregate
4. Determine air entrainment requirements.
5. Evaluate workability requirements of the plastic concrete.
6. Estimate the water content requirements of the mix.
7. Determine cementing materials content and type needed.

8. Evaluate the need and application rate of admixtures.
9. Evaluate fine aggregate requirements.
10. Determine moisture corrections.
11. Make and test trial mixes.

Most concrete supply companies have a wealth of experience about how their materials perform in a variety of applications. This experience, accompanied with reliable test data on the relationship between strength and water–cementitious materials ratio, is the most dependable method for selecting mix proportions. However, understanding the basic principles of mixture design and the proper selection of materials and mixture characteristics is as important as the actual calculation. Therefore, the PCA procedure provides guidelines and can be adjusted to match the experience obtained from local conditions. The PCA mix design steps are discussed next.

1. Strength Requirements Variations in materials, and batching and mixing of concrete results in deviations in the strength of the concrete produced by a plant. Generally, the structural design engineer does not consider this variability when determining the size of the structural members. If the materials engineer provides a material with an average strength equal to the strength specified by the designer, then half of the concrete will be weaker than the specified strength. Obviously, this is undesirable. To compensate for the variance in concrete strength, the materials engineer designs the concrete to have an average strength greater than the strength specified by the structural engineer.

In order to compute the strength requirements for concrete mix design, three quantities must be known:

1. the specified compressive strength f'_c
2. the variability or standard deviation s of the concrete
3. the allowable risk of making concrete with an unacceptable strength

The standard deviation in the strength is determined for a plant by making batches of concrete, testing the strength for many samples, and computing the standard deviation using Equation 1.15 in Chapter 1. The allowable risk has been established by the American Concrete Institute (ACI). One of the risk rules states that there should be less than 10% chance that the strength of a concrete mix is less than the specified strength. Assuming that the concrete strength has a normal distribution, the implication of the ACI rule is that 10% of the area of the distribution must be to the left of f'_c , as shown in Figure 7.1.

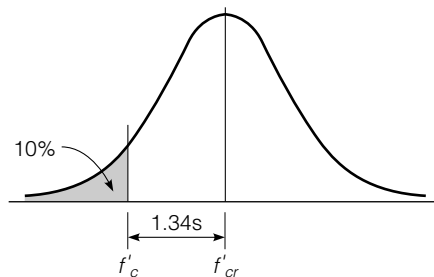


FIGURE 7.1 Use of normal distribution and risk criteria to estimate average required concrete strength.

Using a table of standard z values for a normal distribution curve, we can determine that 90% of the area under the curve will be to the right of f'_c if the average strength is 1.34 standard deviations from f'_c . In other words, the required average strength f'_{cr} for this criterion can be calculated as

$$f'_{cr} = f'_c + 1.34s \tag{7.1}$$

where

- f'_{cr} = required average compressive strength, MPa or psi
- f'_c = specified compressive strength, MPa or psi
- s = standard deviation, MPa or psi

For mixes with a large standard deviation in strength, the ACI has another risk criterion that requires

$$f'_{cr} = f'_c + 2.33s - 3.45 \tag{7.2}$$

The required average compressive strength f'_{cr} is determined as the larger value obtained from Equations 7.1 and 7.2.

Equation 7.2 is valid for SI units only. If U.S. customary units are used, f'_{cr} , f'_c , and s are recorded in psi and the constant 3.45 in Equation 7.2 should be changed to 500.

The standard deviation should be determined from at least 30 strength tests. If the standard deviation is computed from 15 to 30 samples, then the standard deviation is multiplied by the following factor, F , to determine the modified standard deviation s' .

Number of Tests	Modification Factor F
15	1.16
20	1.08
25	1.03
30 or more	1.00

Linear interpolation is used for an intermediate number of tests, and s' is used in place of s in Equations 7.1 and 7.2.

If fewer than 15 tests are available, the following adjustments are made to the specified strength, instead of using Equations 7.1 and 7.2:

Specified Compressive Strength f'_c , MPa (psi)	Required Average Compressive Strength f'_{cr} , MPa (psi)
<21 (<3000)	$f'_c + 7.0$ ($f'_c + 1000$)
21 to 35 (3000 to 5000)	$f'_c + 8.5$ ($f'_c + 1200$)
>35 (>5000)	$f'_c + 10.0$ ($f'_c + 1400$)

These estimates are very conservative and should not be used for large projects, since the concrete will be overdesigned and, therefore, not economical.

Sample Problem 7.1

The design engineer specifies a concrete strength of 31.0 MPa (4500 psi). Determine the required average compressive strength for

- a new plant for which s is unknown
- a plant for which $s = 3.6$ MPa (520 psi) for 17 test results
- a plant with extensive history of producing concrete with $s = 2.4$ MPa (350 psi)
- a plant with extensive history of producing concrete with $s = 3.8$ MPa (550 psi)

Solution

- $f'_{cr} = f'_c + 8.3 = 31.0 + 8.3 = 39.3$ MPa (5700 psi)
- Need to interpolate modification factor:

$$F = 1.16 - \left(\frac{1.16 - 1.08}{20 - 15} \right) (17 - 15) \cong 1.13$$

Multiply standard deviation by the modification factor

$$s' = (s)(F) = 3.6(1.13) = 4.1 \text{ MPa (590 psi)}$$

Determine maximum from Equations 7.1 and 7.2

$$f'_{cr} = 31.0 + 1.34(4.1) = 36.5 \text{ MPa (5300 psi)}$$

$$f'_{cr} = 31.0 + 2.33(4.1) - 3.45 = 37.1 \text{ MPa (5390 psi)}$$

$$\text{Use } f'_{cr} = 37.1 \text{ MPa (5390 psi)}$$

- Determine maximum from Equations 7.1 and 7.2

$$f'_{cr} = 31.0 + 1.34(2.4) = 34.2 \text{ MPa (4970 psi)}$$

$$f'_{cr} = 31.0 + 2.33(2.4) - 3.45 = 33.1 \text{ MPa (4810 psi)}$$

$$\text{Use } f'_{cr} = 34.2 \text{ MPa (4970 psi)}$$

- Determine maximum from Equations 7.1 and 7.2

$$f'_{cr} = 31.0 + 1.34(3.8) = 36.1 \text{ MPa (5240 psi)}$$

$$f'_{cr} = 31.0 + 2.33(3.8) - 3.45 = 36.4 \text{ MPa (5280 psi)}$$

$$\text{Use } f'_{cr} = 36.4 \text{ MPa (5280 psi)}$$

2. Water–Cementitious Materials Ratio Requirements The next step is to determine the water–cementitious materials ratio needed to produce the required

strength. Historical records are used to plot a strength-versus-water-cementitious materials ratio curve, such as that seen in Figure 7.2. If historical data are not available, three trial batches are made at different water-cementitious materials ratios to establish a curve similar to Figure 7.2. Table 7.1 can be used for estimating the water-cementitious materials ratios for the trial mixes when no other data are available. The required average compressive strength is used with the strength versus water-cementitious materials relationship to determine the water-cementitious materials ratio required for the strength requirements of the project.

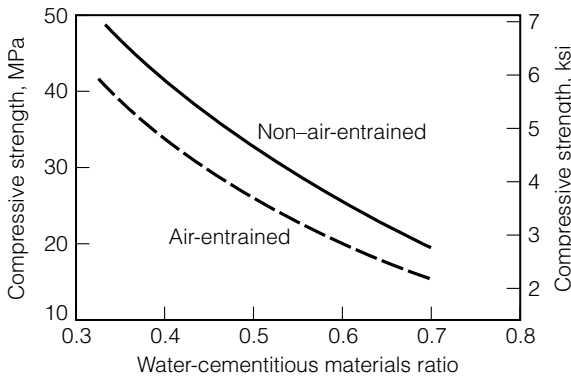


FIGURE 7.2 Example trial mixture or field data strength curves.

TABLE 7.1 Typical Relationship between Water-Cementitious Materials Ratio and Compressive Strength of Concrete*

Weight	Water-Cementitious Materials Ratio by	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
48 (7000)	0.33	—
41 (6000)	0.41	0.32
35 (5000)	0.48	0.40
28 (4000)	0.57	0.48
21 (3000)	0.68	0.59
14 (2000)	0.82	0.74

*American Concrete Institute (ACI 211.1 and ACI 211.3)

**Strength is based on cylinders moist-cured 28 days in accordance with ASTM C31 (AASHTO T23). Relationship assumes nominal maximum size of aggregate about 19 to 25 mm (3/4 to 1 in.).

TABLE 7.2 Maximum Permissible Water–Cementitious Materials Ratios for Concrete when Strength Data from Field Experience or Trial Mixtures are not Available*

Specified 28-day compressive Strength, f'_c , MPa (psi)	Water–Cementitious Materials Ratio by Weight	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
17 (2500)	0.67	0.54
21 (3000)	0.58	0.46
24 (3500)	0.51	0.40
28 (4000)	0.44	0.35
31 (4500)	0.38	**
35 (5000)	**	**

*American Concrete Institute (ACI 318), 1999.

**For strength above 31.0 MPa (4500 psi) (non-air-entrained concrete) and 27.6 MPa (4000 psi) (air-entrained concrete), concrete proportions shall be established from field data or trial mixtures.

For small projects of noncritical applications, Table 7.2 can be used in lieu of trial mixes, with the permission of the project engineer. Table 7.2 is conservative with respect to the strength versus water–cementitious materials ratio relationship. This results in higher cement factors and greater average strengths than would be required if a mix design is performed. This table is not intended for use in designing trial batches; use Table 7.1 for trial batch design.

The water–cementitious materials ratio required for strength is checked against the maximum allowable water–cementitious materials ratio for the exposure conditions. Tables 7.3 and 7.4 provide guidance on the maximum allowable water–cementitious materials ratio and the minimum design compressive strength for exposure conditions. Generally, more severe exposure conditions require lower water–cementitious materials ratios. The minimum of the water–cementitious materials ratio for strength and exposure is selected for proportioning the concrete.

If a pozzolan is used in the concrete, the water–cementitious materials plus pozzolan ratio by weight may be used instead of the traditional water–cementitious materials ratio. In other words, the weight of the water is divided by the sum of the weights of cement plus pozzolan.

3. Coarse Aggregate Requirements The next step is to determine the suitable aggregate characteristics for the project. In general, large dense graded aggregates provide the most economical mix. Large aggregates minimize the amount of water required and, therefore, reduce the amount of cement required per cubic

TABLE 7.3 Maximum Water–Cementitious Material Ratios and Minimum Design Strengths for Various Exposure Conditions*

Exposure Condition	Maximum Water–Cementitious Material Ratio by Mass for Concrete	Minimum Design Compressive Strength, f'_c , MPa (psi)
Concrete protected from exposure to freezing and thawing, application of deicing chemicals, or aggressive substances	Select water–cementitious material ratio on basis of strength, workability, and finishing needs	Select strength based on structural requirements
Concrete intended to have low permeability when exposed to water	0.50	28 (4000)
Concrete exposed to freezing and thawing in a moist condition or deicers	0.45	31 (4500)
For corrosion protection for reinforced concrete exposed to chlorides from deicing salts, salt water, brackish water, seawater, or spray from these sources	0.40	35 (5000)

*American Concrete Institute (ACI 318), 1999.

meter of mix. Round aggregates require less water than angular aggregates for an equal workability.

The maximum allowable aggregate size is limited by the dimensions of the structure and the capabilities of the construction equipment. The largest maximum aggregate size practical under job conditions that satisfies the size limits in the table should be used.

Situation	Maximum Aggregate Size
Form dimensions	1/5 of minimum clear distance
Clear space between reinforcement or prestressing tendons	3/4 of minimum clear space
Clear space between reinforcement and form	3/4 of minimum clear space
Unreinforced slab	1/3 of thickness

TABLE 7.4 Requirements for Concrete Exposed to Sulfates in Soil or Water*

Sulfate Exposure	Water-Soluble Sulfate (SO ₄) in Soil, Percent by Weight**	Sulfate (SO ₄) in Water, ppm**	Cement Type***	Maximum Water–Cementitious Material Ratio by Weight	Minimum Design Compressive Strength, f'_c , MPa (psi)
Negligible	Less than 0.10	Less than 150	No special type required	—	—
Moderate****	0.10–0.20	150–1500	II, MS, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	28 (4000)
Severe	0.20–2.00	1500–10,000	V, HS	0.45	31 (4500)
Very Severe	Over 2.00	Over 10,000	V, HS	0.40	35 (5000)

*Adopted from American Concrete Institute (ACI 318), 1999.

**Tested in accordance with the Method for Determining the Quantity of Soluble Sulfate in Solid (Soil and Rock) and Water Samples, Bureau of Reclamation, Denver, 1977.

***Cement Types II and V are in ASTM C150 (AASHTO M85), Types MS and HS in ASTM C1157, and the remaining types are in ASTM C595 (AASHTO M240). Pozzolans or slags that have been determined by test or severe record to improve sulfate resistance may also be used.

****Sea water.

Sample Problem 7.2

A structure is to be built with concrete with a minimum dimension of 0.2 m, minimum space between rebars of 40 mm, and minimum cover over rebars of 40 mm. Two types of aggregate are locally available, with maximum sizes of 19 mm and 25 mm, respectively. If both types of aggregate have essentially the same cost, which one is more suitable for this structure?

Solution

25 mm < (1/5)(200 mm) minimum dimensions.

25 mm < (3/4)(40 mm) rebar spacing.

25 mm < (3/4)(40 mm) rebar cover.

Therefore, both sizes satisfy the dimension requirements. However, 25 mm aggregate is more suitable, because it will produce more economical concrete mix.

The gradation of the fine aggregates is defined by the fineness modulus. The desirable fineness modulus depends on the coarse aggregate size and the quantity of cement paste. A low fineness modulus is desired for mixes with low cement content to promote workability.

Once the fineness modulus of the fine aggregate and the maximum size of the coarse aggregate are determined, the volume of coarse aggregate per unit volume of concrete is determined using Table 7.5. For example, if the fineness modulus of the fine aggregate is 2.60 and the maximum aggregate size is 25 mm (1 in.), the coarse aggregate will have a volume of 0.69 m³/m³ (yd³/yd³) of concrete. Table 7.5 is based on the unit weight of aggregates in a dry-rodded condition (ASTM C29). The values given are based on experience in producing an average degree of workability. The volume of coarse aggregate can be increased by 10% when less workability is required, such as in pavement construction. The volume of coarse aggregate should be reduced by 10% to increase workability, for example to allow placement by pumping.

4. Air Entrainment Requirements Next, the need for air entrainment is evaluated. Air entrainment is required whenever concrete is exposed to freeze–thaw conditions and deicing salts. Air entrainment is also used for workability in some situations. The amount of air required varies based on exposure conditions

TABLE 7.5 Bulk Volume of Coarse Aggregate per Unit Volume of Concrete*

Nominal Maximum Size of Aggregate, mm (in.)	Bulk Volume of Dry-Rodded Coarse Aggregate Per Unit Volume of Concrete for Different Fineness Moduli of Fine Aggregate**			
	Fineness Modulus			
	2.40	2.60	2.80	3.00
9.5 (3/8)	0.50	0.48	0.46	0.44
12.5 (1/2)	0.59	0.57	0.55	0.53
19 (3/4)	0.66	0.64	0.62	0.60
25 (1)	0.71	0.69	0.67	0.65
37.5 (1 1/2)	0.75	0.73	0.71	0.69
50 (2)	0.78	0.76	0.74	0.72
75 (3)	0.82	0.80	0.78	0.76
150 (6)	0.87	0.85	0.83	0.81

*American Concrete Institute (ACI 211.1).

**Bulk volumes are based on aggregates in a dry-rodded condition as described in ASTM C29 (AASHTO T19).

and is affected by the size of the aggregates. The exposure levels are defined as follows:

- Mild exposure*—Indoor or outdoor service in which concrete is not exposed to freezing and deicing salts. Air entrainment may be used to improve workability.
- Moderate exposure*—Some freezing exposure occurs, but concrete is not exposed to moisture or free water for long periods prior to freezing. Concrete is not exposed to deicing salts. Examples include exterior beams, columns, walls, etc., not exposed to wet soil.
- Severe exposure*—Concrete is exposed to deicing salts, saturation, or free water. Examples include pavements, bridge decks, curbs, gutters, canal linings, etc.

Table 7.6 presents the recommended air contents for different combinations of exposure conditions and maximum aggregate sizes. The values shown in Table 7.6 are the entrapped air for non-air-entrained concrete and the entrapped plus entrained air in case of air-entrained concrete. The recommended air content decreases with increasing maximum aggregate size.

5. Workability Requirements The next step in the mix design is to determine the workability requirements for the project. Workability is defined as the ease of placing, consolidating, and finishing freshly mixed concrete. Concrete should be workable but should not segregate or excessively bleed (migration

TABLE 7.6 Approximate Target Percent Air Content Requirements for Different Nominal Maximum Sizes of Aggregates*

	Maximum Aggregate Size							
	9.5 mm (3/8 in.)	12.5 mm (1/2 in.)	19 mm (3/4 in.)	25 mm (1 in.)	37.5 mm (1 1/2 in.)	50 mm (2 in.)	75 mm (3 in.)	150 mm (6 in.)
Non-air-entrained concrete	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete**								
Mild Exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Moderate Exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
Severe Exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5	4.0

*American Concrete Institute (ACI 211.1 and ACI 318).

**The air content in job specifications should be specified to be delivered within -1 to +2 percentage points of the table target value for moderate and severe exposures.



FIGURE 7.3 Slump test apparatus.

of water to the top surface of concrete). The slump test (Figure 7.3) is an indicator of workability when evaluating similar mixtures. This test consists of filling a truncated cone with concrete, removing the cone, then measuring the distance the concrete slumps (ASTM C143). The slump is increased by adding water, air entrainer, water reducer, superplasticizer, or by using round aggregates. Table 7.7 provides recommendations for the slump of concrete used in different types of projects. For batch adjustments, slump

TABLE 7.7 Recommended Slumps for Various Types of Construction*

Concrete Construction	Slump, mm (in.)	
	Maximum**	Minimum
Reinforced foundation walls and footings	75 (3)	25 (1)
Plain footings, caissons, and substructure walls	75 (3)	25 (1)
Beams and reinforced walls	100 (4)	25 (1)
Building columns	100 (4)	25 (1)
Pavements and slabs	75 (3)	25 (1)
Mass concrete	75 (3)	25 (1)

*American Concrete Institute (ACI 211.1).

**May be increased 25 mm (1 in.) for consolidation by hand methods such as rodding and spading. Plasticizers can safely provide higher slumps.

increases about 25 mm (1in.) for each 6 kg of water added per m^3 (10 lb per cubic yard) of concrete.

6. Water Content Requirements The water content required for a given slump depends on the maximum size and shape of the aggregates and whether an air entrainer is used. Table 7.8 gives the approximate mixing water requirements for angular coarse aggregates (crushed stone). The recommendations in Table 7.8 are reduced for other aggregate shape as shown in this table.

Aggregate Shape	Reduction in Water Content, kg/m^3 (lb/yd ³)
Subangular	12 (20)
Gravel with crushed particles	21 (35)
Round gravel	27 (45)

These recommendations are approximate and should be verified with trial batches for local materials.

7. Cementing Materials Content Requirements With the water–cementitious materials ratio and the required amount of water estimated, the amount of cementing materials required for the mix is determined by dividing the weight of the water by the water–cementitious materials ratio. PCA recommends a minimum cement content of 334 kg/m^3 (564 lb/yd^3) for concrete exposed to severe freeze–thaw, deicers, and sulfate exposures, and not less than 385 kg/m^3 (650 lb/yd^3) for concrete placed under water. In addition, Table 7.9 shows the minimum cement requirements for proper placing, finishing, abrasion resistance, and durability in flatwork, such as slabs.

8. Admixture Requirements If one or more admixtures are used to add a specific quality in the concrete (as discussed in Chapter 6), their quantities should be considered in the mix proportioning. Admixture manufacturers provide specific information on the quantity of admixture required to achieve the desired results.

9. Fine Aggregate Requirements At this point, water, cement, and coarse aggregate weights per cubic meter (cubic yard) are known and the volume of air is estimated. The only remaining factor is the amount of fine aggregates needed. The weight mix design method uses Table 7.10 to estimate the total weight of a “typical” freshly mixed concrete for different maximum aggregate sizes. The weight of the fine aggregates is determined by subtracting the weight of the other ingredients from the total weight. Since Table 7.10 is based on a “typical” mix, the weight-based mix design method is only approximate.

TABLE 7.8 Approximate Mixing Water in kg/m³ (lb/yd³) for Different Slumps and Nominal Maximum Aggregate Sizes*

Slump, mm (in.)	Maximum Aggregate Size in mm (in.)**							
	9.5 (3/8)	12.5 (1/2)	19 (3/4)	25 (1)	37.5 (1 1/2)	50 (2)**	75 (3)***	150 (6)***
	Non-air-entrained concrete							
25 to 50 (1 to 2)	207 (350)	199 (335)	190 (315)	179 (300)	166 (275)	154 (260)	130 (220)	113 (190)
75 to 100 (3 to 4)	228 (385)	216 (365)	205 (340)	193 (325)	181 (300)	169 (285)	145 (245)	124 (210)
150 to 175 (6 to 7)	243 (410)	228 (385)	216 (360)	202 (340)	190 (315)	178 (300)	160 (270)	—
	Air-entrained concrete							
25 to 50 (1 to 2)	181 (305)	175 (295)	168 (280)	160 (270)	150 (250)	142 (240)	122 (205)	107 (180)
75 to 100 (3 to 4)	202 (340)	193 (325)	184 (305)	175 (295)	165 (275)	157 (265)	133 (225)	119 (200)
150 to 175 (6 to 7)	216 (365)	205 (345)	197 (325)	184 (310)	174 (290)	166 (280)	154 (260)	—

*American Concrete Institute (ACI 211.1 and ACI 318).

**These quantities of mixing water are for use in computing cementitious material contents for trial batches. They are maximums for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications.

***The slump values for concrete containing aggregates larger than 37.5 mm (1 1/2 in.) are based on slump tests made after removal of particles larger than 37.5 mm by wet screening.

TABLE 7.9 Minimum Requirements of Cementing Materials for Concrete Used in Flatwork*

Maximum Size of Aggregate mm (in.)	Cementing Materials, kg/m ³ (lb/yd ³)**
37.5 (1½)	280 (470)
25.0 (1)	310 (520)
19.0 (¾)	320 (540)
12.5 (½)	350 (590)
9.5 (3/8)	360 (610)

*American Concrete Institute (ACI 302).

**Cementing materials quantities may need to be greater for severe exposure. For example, for deicer exposures, concrete should contain at least 335 kg/m³ (564 lb/yd³) of cementing materials.

TABLE 7.10 Estimate of Weight of Freshly Mixed Concrete

Maximum Aggregate Size, mm (in.)	Non Air Entrained Concrete kg/m ³ (lb/yd ³)	Air Entrained Concrete kg/m ³ (lb/y ³)
9.5 (3/8)	2276 (3840)	2187 (3690)
12.5 (½)	2305 (3890)	2228 (3760)
19.0 (¾)	2347 (3960)	2276 (3840)
25.0 (1)	2376 (4010)	2311 (3900)
37.5 (1½)	2412 (4070)	2347 (3960)
50.0 (2)	2441 (4120)	2370 (4000)
75.0 (3)	2465 (4160)	2394 (4040)
150 (6)	2507 (4230)	2441 (4120)

In the absolute volume method of mix design, the component weight and the specific gravity are used to determine the volumes of the water, coarse aggregate, and cement. These volumes, along with the volume of the air, are subtracted from a unit volume of concrete to determine the volume

of the fine aggregate required. The volume of the fine aggregate is then converted to a weight using the unit weight. Generally, the bulk SSD specific gravity of aggregates is used for the weight–volume conversions of both fine and coarse aggregates.

10. Moisture Corrections Mix designs assume that water used to hydrate the cement is the free water in excess of the moisture content of the aggregates at the SSD condition (absorption), as discussed in Chapter 5. Therefore, the final step in the mix design process is to adjust the weight of water and aggregates to account for the existing moisture content of the aggregates. If the moisture content of the aggregates is more than the SSD moisture content, the weight of mixing water is reduced by an amount equal to the free weight of the moisture on the aggregates. Similarly, if the moisture content is below the SSD moisture content, the mixing water must be increased.

11. Trial Mixes After computing the required amount of each ingredient, a trial batch is mixed to check the mix design. Three 0.15 m × 0.30 m (6 in. × 12 in.) cylinders are made, cured for 28 days, and tested for compressive strength. In addition, the air content and slump of fresh concrete are measured. If the slump, air content, or compressive strength does not meet the requirements, the mixture is adjusted and other trial mixes are made until the design requirements are satisfied.

Additional trial batches could be made by slightly varying the material quantities in order to determine the most workable and economical mix.

Sample Problem 7.3

You are working on a concrete mix design that requires each cubic yard of concrete to have a 0.43 water–cementitious materials ratio, 2077 lb/yd³ of dry gravel, 244 lb/yd³ of water, and 4% air content. The available gravel has a specific gravity of $G_{\text{gravel}} = 2.6$, a moisture content of 2.3% and absorption of 4.5%. The available sand has a specific gravity of $G_{\text{sand}} = 2.4$, a moisture content of 2.2% and absorption of 1.7%. Air entrainer is to be included using the manufacturers specification of 0.1 fl. oz / 1% air / 100 lb cement.

For each cubic yard of concrete needed on the job, calculate the weight of cement, moist gravel, moist sand, and water that should be added to the batch. Summarize and total the mix design when finished.

Solution

$$\text{Step 7: } W_{\text{cement}} = 244/0.43 = 567 \text{ lb/yd}^3$$

$$\text{Step 8: air entrainer} = (0.1)(4)(567/100) = 2.27 \text{ fl. oz}$$

$$\text{Step 9: } \gamma_w = 62.4 \text{ lb/ft}^3 (3 \text{ ft/yd})^3 = 1684.8 \text{ lb/yd}^3$$

$$\begin{aligned}
 V_{\text{cement}} &= 567.4 / (3.15 \times 1684.8) &= 0.107 \text{ yd}^3 \\
 V_{\text{water}} &= 244 / 1684.8 &= 0.145 \text{ yd}^3 \\
 V_{\text{gravel}} &= 2077 / (2.6 \times 1684.8) &= 0.474 \text{ yd}^3 \\
 V_{\text{air}} &= 4\% &= 0.040 \text{ yd}^3 \\
 &\text{Subtotal} &= 0.766 \text{ yd}^3 \\
 \\
 V_{\text{sand}} &= 1 - 0.766 = 0.234 \text{ yd}^3 \\
 m_{\text{sand}} &= (0.234)(2.4)(1684.8) = 946 \text{ lb/yd}^3
 \end{aligned}$$

Step 10: mix water = $244 - 2077(0.023 - 0.045) - 946.2(0.022 - 0.017)$
 $= 285 \text{ lb/yd}^3$

moist gravel = $2077(1.023)$ = 2125 lb/yd^3
moist sand = $946.2(1.022)$ = 967 lb/yd^3
cement = 567 lb/yd^3
air entrainer = 2.27 fl. oz

Sample Problem 7.4

Design a concrete mix for the following conditions and constraints using the absolute volume method:

Design Environment

Bridge pier exposed to freezing and subjected to deicing chemicals

Required design strength = 24.1 MPa (3500 psi)

Minimum dimension = 0.3 m (12 in.)

Minimum space between rebars = 50 mm (2 in.)

Minimum cover over rebars = 40 mm (1.5 in.)

Standard deviation of compressive strength of 2.4 MPa (350 psi) is expected
(more than 30 samples)

Only air entrainer is allowed

Available Materials

Cement

Select Type V due to exposure

Air Entrainer

Manufacturer specification 6.3 ml/1% air/100 kg cement (0.1 fl oz/1% air/100 lb cement)

Coarse aggregate

25 mm (1 in.) maximum size, river gravel (round)

Bulk oven dry specific gravity = 2.621, Absorption = 0.4%

Oven dry-rodded density = 1681 kg/m³ (105 pcf)

Moisture content = 1.5%

Fine aggregate

Natural sand

Bulk oven-dry specific gravity = 2.572, Absorption = 0.8%

Moisture content = 4%

Fineness modulus = 2.60

Solution

1. Strength Requirements

$s = 2.4 \text{ MPa (350 psi)}$ (enough samples so that no correction is needed)

$$f'_{cr} = f'_c + 1.34s = 24.1 + 1.34(2.4) = 27.3 \text{ MPa (3960 psi)}$$

$$f_{cr} = f_c + 2.33s - 3.45 = 24.1 + 2.33(2.4) - 3.45 = 26.2 \text{ MPa (3810 psi)}$$

$$f'_{cr} = \mathbf{27.3 \text{ MPa (3960 psi)}}$$

2. Water–Cementitious Materials Ratio

Strength requirement (Table 7.1), water–cementitious materials ratio = 0.48 by interpolation

Exposure requirement (Tables 7.3 and 7.4), maximum water–cementitious materials ratio = 0.45

Water–cementitious materials ratio = 0.45

3. Coarse Aggregate Requirements

$$25 \text{ mm} < \frac{1}{5} (300 \text{ mm}) \text{ minimum dimensions}$$

$$25 \text{ mm} < \frac{3}{4} (50 \text{ mm}) \text{ rebar spacing}$$

$$25 \text{ mm} < \frac{3}{4} (40 \text{ mm}) \text{ rebar cover}$$

Aggregate size Okay for dimensions

(Table 7.5) 25 mm maximum size coarse aggregate and 2.60 FM fine aggregate

Coarse aggregate factor = 0.69

$$\text{Dry weight of coarse aggregate} = (1681)(0.69) = 1160 \text{ kg/m}^3 (1956 \text{ lb/yd}^3)$$

Coarse aggregate = 1160 kg/m³ (1956 lb/yd³)

4. Air Content

(Table 7.6) Severe exposure, target air content = 6.0%

Job range = 5% to 8% base

Design using 7%

5. Workability

(Table 7.7) Pier best fits the column requirement in the table

Slump range = 25 to 100 mm (1 to 4 in.)

Use 75 mm (3 in.)

6. Water Content

(Table 7.8) 25 mm aggregate with air entrainment and 75 mm slump

Water = 175 kg/m³ (295 lb/yd³) for angular aggregates. Since we have round coarse aggregates, reduce by 27 kg/m³ (45 lb/yd³)

Required water = 148 kg/m³ (250 lb/yd³)

7. Cementing Materials Content

Water–cementitious materials ratio = 0.45, water = 148 kg/m³ (250 lb/yd³)

Cement = 148/0.45 = 329 kg/m³ (556 lb/yd³)

Increase for minimum criterion of 334 kg/m³ (564 lb/yd³) for exposure

Cement = 334 kg/m³ (564 lb/yd³)

8. Admixture

7% air, cement = 334 kg/m³ (564 lb/yd³)

Admixture = (6.3)(7)(334/100) = 147 ml/m³ (3.9 fl oz/yd³)

Admixture = 147 ml/m³ (3.9 fl oz/yd³)

9. Fine Aggregate Requirements

Find fine aggregate content; use the absolute volume method.

Water volume = 148/(1 × 1000) = 0.148 m³/m³ (4.006 ft³/yd³)

Cement volume = 334/(3.15 × 1000) = 0.106 m³/m³ (2.869 ft³/yd³)

Air volume = 0.07 m³/m³ (0.07 × 27 = 1.890 ft³/yd³)

Coarse aggregate volume = 1160/(2.621 × 1000) = 0.443 m³/m³ (11.960 ft³/yd³)

Subtotal volume = 0.767 m³/m³ (20.725 ft³/yd³)

Fine aggregate

volume = 1 – 0.767 = 0.233 m³/m³ (27 – 20.725 = 6.275 ft³/yd³)

Fine aggregate dry weight = (0.233)(2.572)(1000) = 599 kg/m³ (1007 lb/yd³)

Fine aggregate = 599 kg/m³ (1007 lb/yd³)

10. Moisture Corrections

Coarse aggregate: Need 1160 kg/m³ (1956 lb/yd³) in dry condition, so increase by 1.5% for moisture

Moist coarse aggregate = (1160)(1.015) = 1177 kg/m³ (1985 lb/yd³)

Fine aggregate: Need 599 kg/m³ (1007 lb/yd³) in dry condition, so increase 4% for moisture

Fine aggregate in moist condition = (599)(1.04) = 623 kg/m³ (1047 lb/yd³)

Water: Reduce for free water on aggregates = 148 – 1160(0.015 – 0.004)

– 599(0.04 – 0.008) = 116 kg/m³ (196 lb/yd³)

Summary

**Batch Ingredients
Required**

	1 m ³ PCC	1 yd ³ PCC
Water	116 kg	196 lb
Cement	334 kg	564 lb
Fine aggregate	623 kg	1047 lb
Coarse aggregate	1177 kg	1985 lb
Admixture	147 ml	3.9 fl oz

7.1.2 ■ Mixing Concrete for Small Jobs

The mix design process applies to large jobs. For small jobs, for which a large design effort is not economical (e.g., jobs requiring less than one cubic meter of concrete), Tables 7.11 and 7.12 can be used as a guide. The values in these tables may need to be adjusted to obtain a workable mix, using the

TABLE 7.11 Relative Components of Concrete for Small Jobs, by Weight*

Maximum Size of Coarse Aggregate mm (in.)	Air-Entrained Concrete				Non-Air-Entrained Concrete			
	Wet Cement	Wet Fine Aggregate	Wet Coarse Aggregate**	Water	Wet Cement	Wet Fine Aggregate	Wet Coarse Aggregate**	Water
9.5 (3/8)	0.210	0.384	0.333	0.073	0.200	0.407	0.317	0.076
12.5 (1/2)	0.195	0.333	0.399	0.073	0.185	0.363	0.377	0.075
19 (3/4)	0.176	0.296	0.458	0.070	0.170	0.320	0.442	0.068
25 (1)	0.169	0.275	0.493	0.063	0.161	0.302	0.470	0.067
37.5 (1 1/2)	0.159	0.262	0.517	0.062	0.153	0.287	0.500	0.060

*Portland Cement Association, 2002.

**If crushed stone is used, decrease coarse aggregate by 50 kg and increase fine aggregate by 50 kg for each cubic meter of concrete (or decrease coarse aggregate by 3 lb and increase fine aggregate by 3 lb for each cubic foot of concrete).

TABLE 7.12 Relative Components of Concrete for Small Jobs, by Volume*

Maximum Size of Coarse Aggregate mm (in.)	Air-Entrained Concrete				Non-Air-Entrained Concrete			
	Wet Cement	Wet Fine Aggregate	Wet Coarse Aggregate	Water	Wet Cement	Wet Fine Aggregate	Wet Coarse Aggregate	Water
9.5 (3/8)	0.190	0.429	0.286	0.095	0.182	0.455	0.272	0.091
12.5 (1/2)	0.174	0.391	0.348	0.087	0.167	0.417	0.333	0.083
19 (3/4)	0.160	0.360	0.400	0.080	0.153	0.385	0.385	0.077
25 (1)	0.154	0.346	0.423	0.077	0.148	0.370	0.408	0.074
37.5 (1 1/2)	0.148	0.333	0.445	0.074	0.143	0.357	0.429	0.071

*Portland Cement Association, 2002.

The combined volume is approximately 2/3 of the sum of the original bulk volumes.

locally available aggregates. Recommendations related to exposure conditions discussed earlier should be followed.

Tables 7.11 and 7.12 are used for proportioning concrete mixes by weight and volume, respectively. The tables provide ratios of components, with a sum of one unit. Therefore, the required total weight or volume of the concrete mix can be multiplied by the given ratios to obtain the weight or volume of each component. Note that for proportioning by volume, the combined volume is approximately two-thirds of the sum of the original bulk volumes of the components, since water and fine materials fill the voids between coarse materials.

Sample Problem 7.5

Determine the required weights of ingredients to make a 3500-lb batch of non-air-entrained concrete mix with a maximum gravel size of 1/2 in.

Solution

From Table 7.11:

$$\text{Weight of cement} = 3500 \times 0.185 = 647.5 \text{ lb}$$

$$\text{Weight of wet fine aggregate} = 3500 \times 0.363 = 1270.5 \text{ lb}$$

$$\text{Weight of wet coarse aggregate} = 3500 \times 0.377 = 1319.5 \text{ lb}$$

$$\text{Weight of water} = 3500 \times 0.075 = 262.5 \text{ lb}$$

Sample Problem 7.6

Determine the required volumes of ingredients to make a 0.5-m³ batch of air-entrained concrete mix with a maximum gravel size of 19 mm.

Solution

Sum of the original bulk volumes of the components = $0.5 \times 1.5 = 0.75 \text{ m}^3$. From Table 7.12:

$$\text{Volume of cement} = 0.75 \times 0.160 = 0.12 \text{ m}^3$$

$$\text{Volume of wet fine aggregate} = 0.75 \times 0.360 = 0.27 \text{ m}^3$$

$$\text{Volume of wet coarse aggregate} = 0.75 \times 0.400 = 0.3 \text{ m}^3$$

$$\text{Volume of water} = 0.75 \times 0.080 = 0.06 \text{ m}^3$$

7.2 Mixing, Placing, and Handling Fresh Concrete

The proper batching, mixing, and handling of fresh concrete are important prerequisites for strong and durable concrete structures. Next we will discuss the basic steps and precautions to be followed in mixing and handling fresh concrete (Mehta and Monteiro 1993; American Concrete Institute 1982; American Concrete Institute 1983).

Batching is measuring and introducing the concrete ingredients into the mixer. Batching by weight is more accurate than batching by volume, since weight batching avoids the problem created by bulking of damp sand. Water and liquid admixtures, however, can be measured accurately either by weight or volume. On the other hand, batching by volume is commonly used with continuous mixers and when hand mixing.

Concrete should be mixed thoroughly, either in a mixer or by hand, until it becomes uniform in appearance. Hand mixing is usually limited to small jobs or situations in which mechanical mixers are not available. Mechanical mixers include on-site mixers and central mixers in ready-mix plants. The capacity of these mixers varies from 1.5 m^3 to 9 m^3 (2 yd^3 to 12 yd^3). Mixers also vary in type, such as tilting, nontilting, and pan-type mixers. Most of the mixers are batch mixers, although some mixers are continuous.

Mixing time and number of revolutions vary with the size and type of the mixer. Specifications usually require a minimum of 1 minute of mixing for stationary mixers of up to 0.75 m^3 (1 yd^3) of capacity, with an increase of 15 seconds for each additional 0.75 m^3 of capacity. Mixers are usually charged with 10% of the water, followed by uniform additions of solids and 80% of the water. Finally, the remainder of the water is added to the mixer.

7.2.1 ■ Ready-Mixed Concrete

Ready-mixed concrete is mixed in a central plant, and delivered to the job site in mixing trucks ready for placing (Figure 7.4). Three mixing methods can be used for ready mixed concrete:

1. Central-mixed concrete is mixed completely in a stationary mixer and delivered in an agitator truck (2 rpm to 6 rpm).
2. Shrink-mixed concrete is partially mixed in a stationary mixer and completed in a mixer truck (4 rpm to 16 rpm).
3. Truck-mixed concrete is mixed completely in a mixer truck (4 rpm to 16 rpm).

Truck manufacturers usually specify the speed of rotation for their equipment. Also, specifications limit the number of revolutions in a truck



FIGURE 7.4 Concrete ready mix plant.

mixer in order to avoid segregation. Furthermore, the concrete should be discharged at the job site within 90 minutes from the start of mixing, even if retarders are used (ASTM C94).

7.2.2 ■ Mobile Batcher Mixed Concrete

Concrete can be mixed in a mobile batcher mixer at the job site (Figure 7.5). Aggregate, cement, water, and admixtures are fed continuously by volume, and the concrete is usually pumped into the forms.

7.2.3 ■ Depositing Concrete

Several methods are available to deposit concrete at the jobsite. Concrete should be deposited continuously as close as possible to its final position. Advance planning and good workmanship are essential to reduce delay, early stiffening and drying out, and segregation. Figures 7.6–7.9 show different methods used to deposit concrete at the jobsite.



FIGURE 7.5 Mobile batcher mixer at the job site.



FIGURE 7.6 Loading concrete in a wheelbarrow.



FIGURE 7.7 Pouring concrete slab.

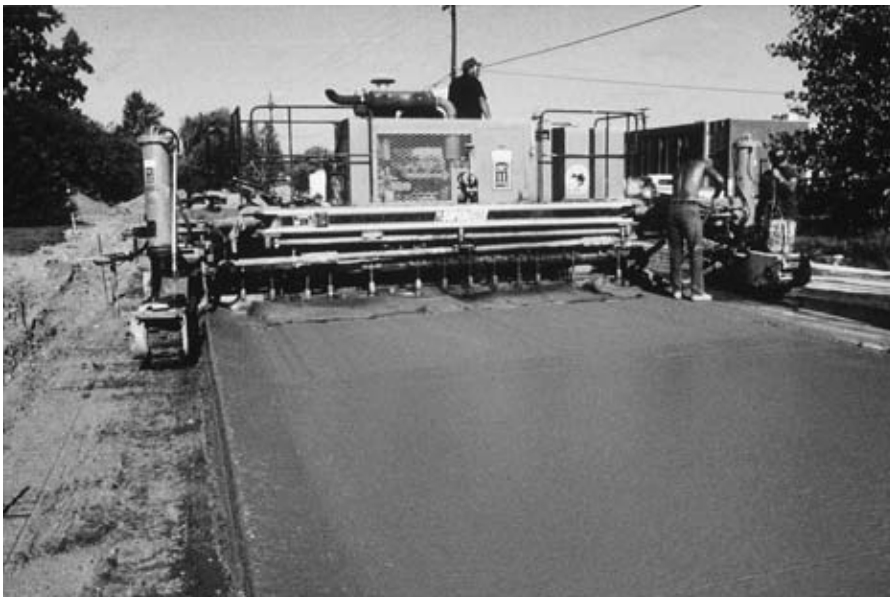


FIGURE 7.8 Placing concrete pavement with a slip-form paver.



FIGURE 7.9 Depositing concrete using a 2-1/2 cubic yard bucket.

7.2.4 ■ Pumped Concrete

Pumped concrete is frequently used for large construction projects. Special pumps deliver the concrete directly into the forms (see Figure 7.10). Careful attention must be exercised to ensure well-mixed concrete with proper workability. The slump should be between 40 mm to 100 mm (1-1/2 in. to 4 in.) before pumping. During pumping, the slump decreases by about 12 mm to 25 mm (1/2 in. to 1 in.), due to partial compaction. Blockage could happen during pumping, due to either the escape of water through the voids in the mix or due to friction if fines content is too high (Neville 1981).

7.2.5 ■ Vibration of Concrete

Quality concrete requires thorough consolidation to reduce the entrapped air in the mix. On small jobs, consolidation can be accomplished manually by ramming and tamping the concrete. For large jobs, vibrators are used to consolidate the concrete. Several types of vibrators are available, depending



FIGURE 7.10 Pumping concrete in a retaining wall.

on the application. *Internal vibrators* are the most common type used on construction projects (see Figure 7.11). These consist of an eccentric weight housed in a spud. The weight is rotated at high speed to produce vibration. The spud is slowly lowered into and through the entire layer of concrete, penetrating into the underlying layer if it is still plastic. The spud is left in place for 5 seconds to 2 minutes, depending on the type of vibrator and the consistency of the concrete. The operator judges the total vibration time required. Over-vibration causes segregation as the mortar migrates to the surface.

Several specialty types of vibrators are used in the production of precast concrete. These include *external vibrators*, *vibrating tables*, *surface vibrators*, *electric hammers*, and *vibratory rollers* (Neville 1981).

7.2.6 ■ Pitfalls and Precautions for Mixing Water

Since the water–cementitious materials ratio plays an important role in concrete quality, the water content must be carefully controlled in the field. Water should not be added to the concrete during transportation. Crews frequently want to increase the amount of water in order to improve workability. If water is added, the hardened concrete will suffer serious loss in quality and strength. The engineer in the field must prevent any attempt to



FIGURE 7.11 Consolidating concrete with an internal vibrator.

increase the amount of mixing water in the concrete beyond that which is specified in the mix design.

7.2.7 ■ Measuring Air Content in Fresh Concrete

Mixing and handling can significantly alter the air content of fresh concrete. Thus, field tests are used to ensure that the concrete has the proper air content prior to placing. Air content can be measured with the pressure, volumetric, gravimetric, or Chace air indicator methods.

The pressure method (ASTM C231) is widely used, since it takes less time than the volumetric method. The pressure method is based on Boyle's law, which relates pressure to volume. A calibrated cylinder (Figure 7.12) is filled with fresh concrete. The vessel is capped and air pressure is applied. The applied pressure compresses the air in the voids of the concrete. The volume of air voids is determined by measuring the amount of volume reduced by the pressure applied. This method is not valid for concrete made with lightweight aggregates, since air in the aggregate voids is also compressed, confounding the measurement of the air content of the cement paste.

The volumetric method for determining air content (ASTM C173) can be used for concrete made with any type of aggregate. The basic process involves placing concrete in a fixed volume cylinder, as shown in Figure 7.13. An equal volume of water is added to the container. Agitation of the container allows the excess water to displace the air in the cement paste voids.

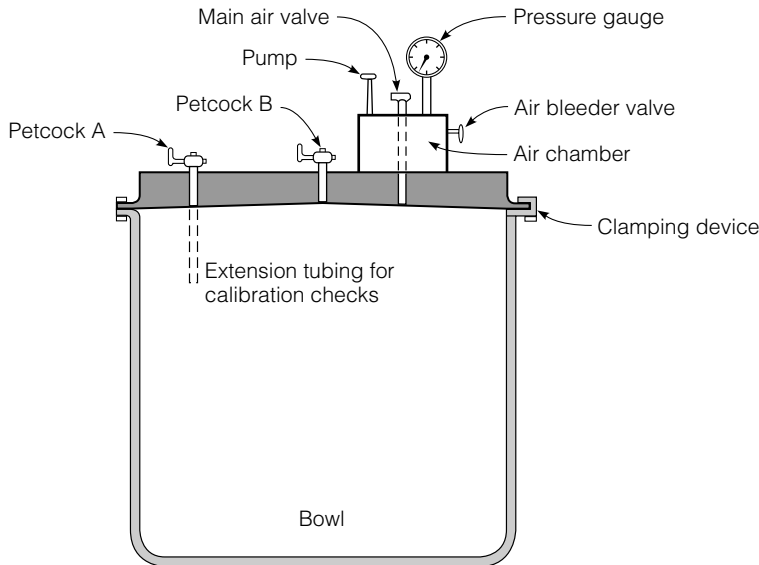


FIGURE 7.12 Pressure method apparatus for determining air voids in fresh concrete – Type B Meter (ASTM C231). (Copyright ASTM. Reprinted with permission.)

The water level in the container falls as the air rises to the top of the container. Thus, the volume of air in the cement paste is directly measured. The accuracy of the method depends on agitating the sample enough to remove all the air from it.

The gravimetric method (ASTM C138) compares the unit weight of freshly mixed concrete with the theoretical maximum unit weight of the mix. The theoretical unit weight is computed from the mix proportions and

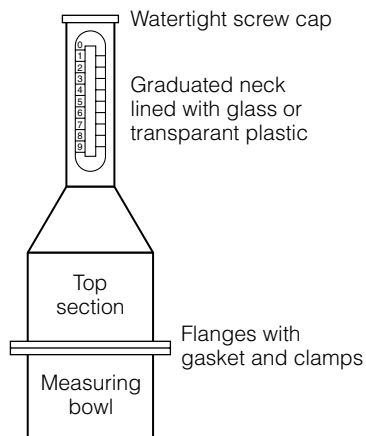


FIGURE 7.13 Volumetric method (Roll-A-Meter) apparatus for determining air voids in fresh concrete (ASTM C173). (Copyright ASTM. Reprinted with permission.)

**FIGURE 7.14** Chace air indicator.

the specific gravity of each ingredient. This method requires very accurate specific gravity measurements, and thus is more suited to the laboratory rather than the field.

The Chace air indicator test (AASHTO T199) is a quick method used to determine the air content of freshly mixed concrete. The device consists of a small glass tube with a stem, a rubber stopper, and a metal cup mounted on the stopper, as shown in Figure 7.14. The metal cup is filled with cement mortar from the concrete to be tested. The indicator is filled with alcohol to a specified level, and the stopper is inserted into the indicator. The indicator is then closed with a finger and gently rolled and tapped until all of the mortar is dispersed in the alcohol and all of the air is displaced with alcohol. With the indicator held in a vertical position, the alcohol level in the stem is read. This reading is then adjusted using calibration tables or figures to determine the air content. The Chace air indicator test can be used to rapidly monitor air content, but it is not precise, nor does it have the repeatability required for specification control. It is especially useful for measuring the air content of small areas near the surface that may have lost air content by improper finishing.

These methods of measuring air content determine the total amount of air, including entrapped air and entrained air, as well as air voids in aggregate particles. Only minute bubbles produced by air-entraining agents impart durability to the concrete. However, the current state of the art is unable to distinguish between the types of air in fresh concrete.

7.2.8 ■ Spreading and Finishing Concrete

Different methods are available to spread and finish concrete, depending on the nature of the structure and the available equipment. Tools and equipment used for spreading and finishing concrete include hand floats, power floats, darbies, bullfloats, straightedges, trowels, vibratory screed, and slip forms. (See Figures 7.8, 7.15–7.18).



FIGURE 7.15 Spreading concrete with a straightedge.



FIGURE 7.16 Finishing concrete with a straightedge.



FIGURE 7.17 Finishing concrete with a power float.



FIGURE 7.18 Finishing concrete with a laser level.

7.3 Curing Concrete

Curing is the process of maintaining satisfactory moisture content and temperature in the concrete for a definite period of time. Hydration of cement is a long-term process and requires water and proper temperature. Therefore, curing allows continued hydration and, consequently, continued gains in concrete strength. In fact, once curing stops, the concrete dries out, and the strength gain stops, as indicated in Figure 7.19. If the concrete is not cured and is allowed to dry in air, it will gain only about 50% of the strength of continuously cured concrete. If concrete is cured for only three days, it will reach about 60% of the strength of continuously cured concrete; if it is cured for seven days, it will reach 80% of the strength of continuously cured concrete. If curing stops for some time and then resumes again, the strength gain will also stop and reactivate.

Increasing temperature increases the rate of hydration and, consequently, the rate of strength development. Temperatures below 10°C (50°F) are unfavorable for hydration and should be avoided, if possible, especially at early ages.

Although concrete of high strength may not be needed for a particular structure, strength is usually emphasized and controlled since it is an indication of the concrete quality. Thus, proper curing not only increases strength, but also provides other desirable properties such as durability, water tightness, abrasion resistance, volume stability, resistance to freeze and thaw, and resistance to deicing chemicals.

Curing should start after the final set of the cement. If concrete is not cured after setting, concrete will shrink, causing cracks. Drying shrinkage can be prevented if ample water is provided for a long period of time. An example of improper curing would be a concrete floor built directly over the

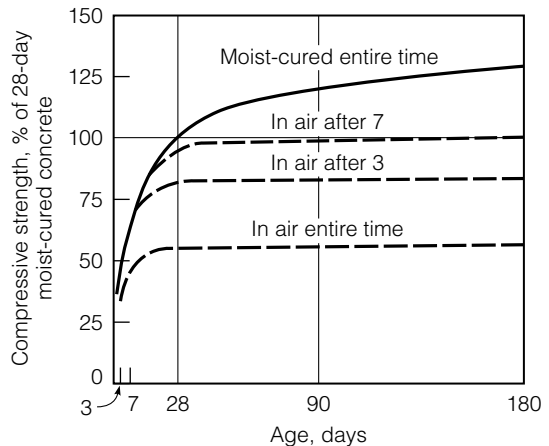


FIGURE 7.19 Compressive strength of concrete at different ages and curing levels.

subgrade, not cured at the surface, with the moisture in the soil curing it from the bottom. In this case the concrete slab may curl due to the relative difference in shrinkage.

Curing can be performed by any of the following methods:

1. maintaining the presence of water in the concrete during early ages. Methods to maintain the water pressure include pounding or immersion, spraying or fogging, and wet coverings.
2. preventing loss of mixing water from the concrete by sealing the surface. Methods to prevent water loss include impervious papers or plastic sheets, membrane-forming compounds, and leaving the forms in place.
3. accelerating the strength gain by supplying heat and additional moisture to the concrete. Accelerated curing methods include steam curing, insulating blankets or covers, and various heating techniques.

Note that preventing loss of mixing water from the concrete by sealing the surface is not as effective as maintaining the presence of water in the concrete during early ages. The choice of the specific curing method or combination of methods depends on the availability of curing materials, size and shape of the structure, in-place versus plant production, economics, and aesthetics (Kosmatka et al. 2002; American Concrete Institute 1986a).

7.3.1 ■ Ponding or Immersion

Ponding involves covering the exposed surface of the concrete structure with water. Ponding can be achieved by forming earth dikes around the concrete surface to retain water. This method is suitable for flat surfaces such as floors and pavements, especially for small jobs. The method requires intensive labor and supervision. Immersion is used to cure test specimens in the laboratory, as well as other concrete members, as appropriate.

7.3.2 ■ Spraying or Fogging

A system of nozzles or sprayers can be used to provide continuous spraying or fogging (see Figures 7.20 and 7.21). This method requires a large amount of water and could be expensive. It is most suitable in high temperature and low humidity environments. Commercial test laboratories generally have a controlled temperature and humidity booth for curing specimens.

7.3.3 ■ Wet Coverings

Moisture-retaining fabric coverings saturated with water, such as burlap, cotton mats, and rugs are used in many applications (see Figure 7.22). The fabric can be kept wet, either by periodic watering or covering the fabric with



FIGURE 7.20 Curing concrete by spraying.



FIGURE 7.21 Curing concrete by fogging.

polyethylene film to retain moisture. On small jobs, wet coverings of earth, sand, saw dust, hay, or straw can be used. Stains or discoloring of concrete could occur with some types of wet coverings.



FIGURE 7.22 Curing concrete by wet covering.

7.3.4 ■ Impervious Papers or Plastic Sheets

Evaporation of moisture from concrete can be reduced using impervious papers, such as kraft papers, or plastic sheets, such as polyethylene film (see Figures 7.23 and 7.24). Impervious papers are suitable for horizontal surfaces



FIGURE 7.23 Curing concrete with impervious fabrics.



FIGURE 7.24 Curing concrete with plastic sheets.

and simply shaped concrete structures, while plastic sheets are effective and easily applied to various shapes. Periodic watering is not required when impervious papers or plastic sheets are used. Discoloration, however, can occur on the concrete surface.

7.3.5 ■ Membrane-Forming Compounds

Various types of liquid membrane-forming compounds can be applied to the concrete surface to reduce or retard moisture loss. These can be used to cure fresh concrete, as well as to hardened concrete, after removal of forms or after moist curing. Curing compounds can be applied by hand or by using spray equipment (see Figures 7.25 and 7.26). Either one coat or two coats (applied perpendicular to each other) are used. Normally, the concrete surface should be damp when the curing compound is applied. Curing compounds should not be used when subsequent concrete layers are to be placed, since the compound hinders the bond between successive layers. Also, some compounds affect the bond between the concrete surface and paint.



FIGURE 7.25 Curing concrete by manually applying membrane forming compound.



FIGURE 7.26 Curing concrete by machine applying membrane forming compound.

7.3.6 ■ Forms Left in Place

Loss of moisture can be reduced by leaving the forms in place as long as practical, provided that the top concrete exposed surface is kept wet. If wood forms are used, the forms should also be kept wet. After removing the forms, another curing method can be used.

7.3.7 ■ Steam Curing

Steam curing is used when early strength gain in concrete is required or additional heat is needed during cold weather. Steam curing can be attained either with or without pressure. Steam at atmospheric pressure is used for enclosed cast-in-place structures and large precast members. High-pressure steam in autoclaves can be used at small manufactured plants.

7.3.8 ■ Insulating Blankets or Covers

When the temperature falls below freezing, concrete should be insulated using layers of dry, porous material such as hay or straw. Insulating blankets manufactured of fiberglass, cellulose fibers, sponge rubber, mineral wool, vinyl foam, or open-cell polyurethane foam can be used to insulate formwork. Moisture proof commercial blankets can also be used.

7.3.9 ■ Electrical, Hot Oil, and Infrared Curing

Precast concrete sections can be cured using electrical, oil, or infrared curing techniques. Electrical curing includes electrically heated steel forms, and electrically heated blankets. Reinforcing steel can be used as a heating element, and concrete can be used as the electrical conductor. Steel forms can also be heated by circulating hot oil around the outside of the structure. Infrared rays have been used for concrete curing on a limited basis.

7.3.10 ■ Curing Period

The curing period should be as long as is practical. The minimum time depends on several factors, such as type of cement, mixture proportions, required strength, ambient weather, size and shape of the structure, future exposure conditions, and method of curing. For most concrete structures the curing period at temperatures above 5°C (40°F) should be a minimum of seven days or until 70% of specified compressive or flexure strength is attained. The curing period can be reduced to three days if high early strength concrete is used and the temperature is above 10°C (50°F).

7.4 Properties of Hardened Concrete

It is important for the engineer to understand the basic properties of hardened portland cement concrete and to be able to evaluate these properties. The main properties of hardened concrete that are of interest to civil and construction engineers include the early volume change, creep, permeability, and stress–strain relation.

7.4.1 ■ Early Volume Change

When the cement paste is still plastic it undergoes a slight decrease in volume of about 1%. This shrinkage is known as *plastic shrinkage* and is due to the loss of water from the cement paste, either from evaporation or from suction by dry concrete below the fresh concrete. Plastic shrinkage may cause cracking (Figure 7.27); it can be prevented or reduced by controlling water loss.

In addition to the possible decrease in volume when the concrete is still plastic, another form of volume change may occur after setting, especially at early ages. If concrete is not properly cured and is allowed to dry, it will shrink. This shrinkage is referred to as *drying shrinkage*, and it also causes cracks. Shrinkage takes place over a long period of time, although the rate of shrinkage is high early, then decreases rapidly with time. In fact, about 15% to 30% of the shrinkage occurs in the first two weeks, while 65% to 85% occurs in the first year. Shrinkage and shrinkage-induced cracking are increased by several factors, including lack of curing, high water–cementitious



FIGURE 7.27 Plastic shrinkage cracking.

materials ratio, high cement content, low coarse aggregate content, existence of steel reinforcement, and aging. On the other hand, if concrete is cured continuously in water after setting, concrete will swell very slightly due to the absorption of water. Since swelling, if it happens, is very small, it does not cause significant problems. *Swelling* is accompanied by a slight increase in weight (Neville 1981).

How much drying shrinkage occurs depends on the size and shape of the concrete structure. Also, nonuniform shrinkage could happen due to the nonuniform loss of water. This may happen in mass concrete structures, where more water is lost at the surface than at the interior. In cases such as this, cracks may develop at the surface. In other cases, curling might develop due to the nonuniform curing throughout the structure and, consequently, nonuniform shrinkage.

7.4.2 ■ Creep Properties

Creep is defined as the gradual increase in strain, with time, under sustained load. Creep of concrete is a long-term process, and it takes place over many years. Although the amount of creep in concrete is relatively small, it could affect the performance of structures. The effect of creep varies with the type of structure. In simply supported reinforced concrete beams, creep increases the deflection and, therefore, increases the stress in the steel. In reinforced concrete columns, creep results in a gradual transfer of load from the concrete to the steel. Creep also could result in losing some of the prestress in prestressed concrete structures, although the use of high-tensile stress steel reduces this effect. Rheological models, discussed in Chapter 1, have been used to analyze the creep response of concrete (Neville 1981).

7.4.3 ■ Permeability

Permeability is an important factor that largely affects the durability of hardened concrete. Permeable concrete allows water and chemicals to penetrate, which, in turn, reduces the resistance of the concrete structure to frost, alkali–aggregate reactivity, and other chemical attacks. Water that permeates into reinforced concrete causes corrosion of steel rebars. Furthermore, impervious concrete is a prerequisite in watertight structures, such as tanks and dams.

Typically, the air voids in the cement paste and aggregates are small and do not affect permeability. However, the air voids that do affect permeability of hardened concrete are obtained from two main sources: incomplete consolidation of fresh concrete and voids resulting from evaporation of mixing water that is not used for hydration of cement.

Therefore, increasing the water–cementitious materials ratio in fresh concrete has a severe effect on permeability. Figure 7.28 shows the typical relationship between the water–cementitious materials ratio and the coefficient of permeability of mature cement paste (Powers 1954). It can be seen

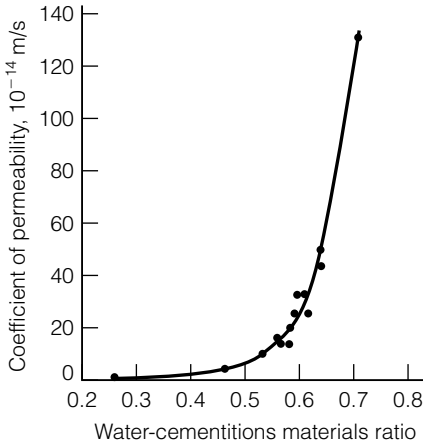


FIGURE 7.28 Relation between water-cementitious materials ratio and permeability of mature cement paste.

from the figure that increasing the water–cementitious materials ratio from 0.3 to 0.7 increases the coefficient of permeability by a factor of 1000. For a concrete to be watertight, the water–cementitious materials ratio should not exceed 0.48 for exposure to fresh water and should not be more than 0.44 for exposure to seawater (American Concrete Institute 1975).

Other factors that affect the permeability include age of concrete, fineness of cement particles, and air-entraining agents. Age reduces the permeability, since hydration products fill the spaces between cement grains. The finer the cement particles, the faster is the rate of hydration and the faster is the development of impermeable concrete. Air-entraining agents indirectly reduce the permeability, since they allow the use of a lower water–cementitious materials ratio.

7.4.4 ■ Stress–Strain Relationship

Typical stress–strain behavior of 28-day-old concrete with different water–cementitious materials ratios are shown in Figure 7.29 (Hognestad et al. 1955). It can be seen that increasing the water–cementitious materials ratio decreases both strength and stiffness of the concrete. The figure also shows that the stress–strain behavior is close to linear at low stress levels, then becomes nonlinear as stress increases. With a water–cementitious materials ratio of 0.50 or less and a strain of up to 0.0015, the stress–strain behavior is almost linear. With higher water–cementitious materials ratios, the stress–strain behavior becomes nonlinear at smaller strains. The curves also show that high-strength concrete has sharp peaks and sudden failure characteristics when compared to low-strength concrete.

As discussed in Chapter 1, the elastic limit can be defined as the largest stress that does not cause a measurable permanent strain. When the concrete is loaded slightly beyond the elastic range and then unloaded, a small amount of strain might remain initially, but it may recover eventually due to creep. Also, since concrete is not perfectly elastic, the rate of loading affects

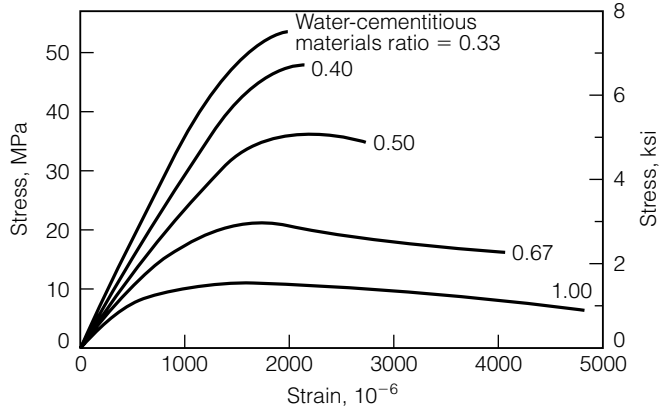


FIGURE 7.29 Typical stress–strain relations for compressive Tests on 0.15 x 0.30-m concrete cylinders at an age of 28 days.

the stress–strain relation to some extent. Therefore, a specific rate of loading is required for testing concrete. It is interesting to note that the shape of the stress–strain relationship of concrete is almost the same for both compression and tension, although the tensile strength is much smaller than the compressive strength. In fact, the tensile strength of concrete typically is ignored in the design of concrete structures.

The modulus of elasticity of concrete is commonly used in designing concrete structures. Since the stress–strain relationship is not exactly linear, the classic definition of modulus of elasticity (Young’s modulus) is not applicable. The initial tangent modulus of concrete has little practical importance. The tangent modulus is valid only for a low stress level where the tangent is determined. Both secant and chord moduli represent “average” modulus values for certain stress ranges. The chord modulus (referred to as the modulus of elasticity) in compression is more commonly used for concrete and is determined according to ASTM C469. The method requires three or four loading and unloading cycles, after which the chord modulus is determined between a point corresponding to a very small strain value and a point corresponding to either 40% of the ultimate stress or a specific strain value. Normal-weight concrete has a modulus of elasticity of 14 GPa to 40 GPa (2,000 ksi to 6,000 ksi).

Poisson’s ratio can also be determined using ASTM C469. Poisson’s ratio is used in advanced structural analysis of shell roofs, flat-plate roofs, and mat foundations. Poisson’s ratio of concrete varies between 0.11 and 0.21, depending on aggregate type, moisture content, concrete age, and compressive strength. A value of 0.15 to 0.20 is commonly used.

It is interesting to note that both aggregate and cement paste, when tested individually, exhibit linear stress–strain behavior. However, the stress–strain relation of concrete is nonlinear. The reason for this behavior is attributed to the microcracking in concrete at the interface between aggregate particles and the cement paste (Shah and Winter 1968).

The modulus of elasticity of concrete increases when the compressive strength increases, as demonstrated in Figure 7.29. There are several empirical

relations between the modulus of elasticity of concrete and the compressive strength. For normal-weight concrete, the relationship used in the United States for designing concrete structures is defined by the ACI Building Code as

$$E_c = 4,731\sqrt{f'_c} \quad (7.3a)$$

or

$$E_c = 57,000\sqrt{f'_c} \quad (7.3b)$$

where

E_c = the modulus of elasticity,
 f'_c = the compressive strength.

Equation 7.3a is used for SI units, where both E_c and f'_c are in MPa, whereas Equation 7.3b is used for the U.S. customary units, where both E_c and f'_c are in psi. This relation is useful, since it relates the modulus of elasticity (needed for designing concrete structures) with the compressive strength, which can be measured easily in the laboratory.

Sample Problem 7.7

A normal-weight concrete has an average compressive strength of 30 MPa. What is the estimated modulus of elasticity?

Solution

$$E_c = 4731\sqrt{f'_c} = 4731(30)^{1/2} = 25,913 \text{ MPa} = 25.9 \text{ GPa}$$

7.5 Testing of Hardened Concrete

Many tests are used to evaluate the hardened concrete properties, either in the laboratory or in the field. Some of these tests are destructive, while others are nondestructive. Tests can be performed for different purposes; however, they are mostly conducted to control the quality of the concrete and to check specification compliance. Probably the most common test performed on hardened concrete is the compressive strength test, since it is relatively easy to perform and since there is a strong correlation between the compressive strength and many desirable properties (Neville 1981; Mehta and Monteiro 1993). Other tests include split tension, flexure strength, rebound hammer, penetration resistance, ultrasonic pulse velocity, and maturity tests.

7.5.1 ■ Compressive Strength Test

The compressive strength test is the test most commonly performed on hardened concrete. Compressive strength is one of the main structural design requirements to ensure that the structure will be able to carry the intended load. As indicated earlier, compressive strength increases as the water–cementitious materials ratio decreases. Since the water–cementitious materials ratio is directly related to the concrete quality, compressive strength is also used as a measure of quality, such as durability and resistance to weathering. Thus, in many cases, designers specify a high compressive strength of the concrete to ensure high quality, even if this strength is not needed for structural support. The compressive strength f'_c of normal-weight concrete is between 20 MPa to 40 MPa (3000 psi to 6000 psi). In the United States, the test is performed on cylindrical specimens and is standardized by ASTM C39. The specimen is prepared, either in the lab or in the field, according to ASTM C192 or C31, respectively. Cores could also be drilled from the structure following ASTM C42. The standard specimen size is 0.15 m (6 in.) in diameter and 0.30 m (12 in.) high, although other sizes with a height–diameter ratio of two can also be used. The diameter of the specimen must be at least three times the nominal maximum size of the coarse aggregate in the concrete.

In the lab, specimens are prepared in three equal layers and are rodded 25 times per layer. After the surface is finished, specimens are kept in the mold for the first 24 ± 8 hours. Specimens are then removed from the mold and cured at $23 \pm 1.7^\circ\text{C}$ ($73.4 \pm 3^\circ\text{F}$), either in saturated-lime water or in a moist cabinet having a relative humidity of 95% or higher, until the time of testing. Before testing, specimens are capped at the two bases to ensure parallel surfaces. High-strength gypsum plaster, sulfur mortar, or a special capping compound can be used for capping and is applied with a special alignment device (ASTM C617). Using a testing machine, specimens are tested by applying axial compressive load with a specified rate of loading until failure (Figure 7.30). The compressive strength of the specimen is determined by dividing the maximum load carried by the specimen during the test by the average cross-sectional area. The number of specimens and the number of test batches depend on established practice and the nature of the test program. Usually three or more specimens are tested for each test age and test condition. Test ages often used are 7 days and 28 days.

Note that the test specimen must have a height–diameter ratio of two. The main reason for this requirement is to eliminate the end effect due to the friction between the loading heads and the specimen. Thus, we can guarantee a zone of uniaxial compression within the specimen. If the height–diameter ratio is less than two, a correction factor can be applied to the results as indicated in ASTM C39.

The compressive strength of the specimen is affected by the specimen size. Increasing the specimen size reduces the strength, because there is a greater probability of weak elements where failure starts in large specimens

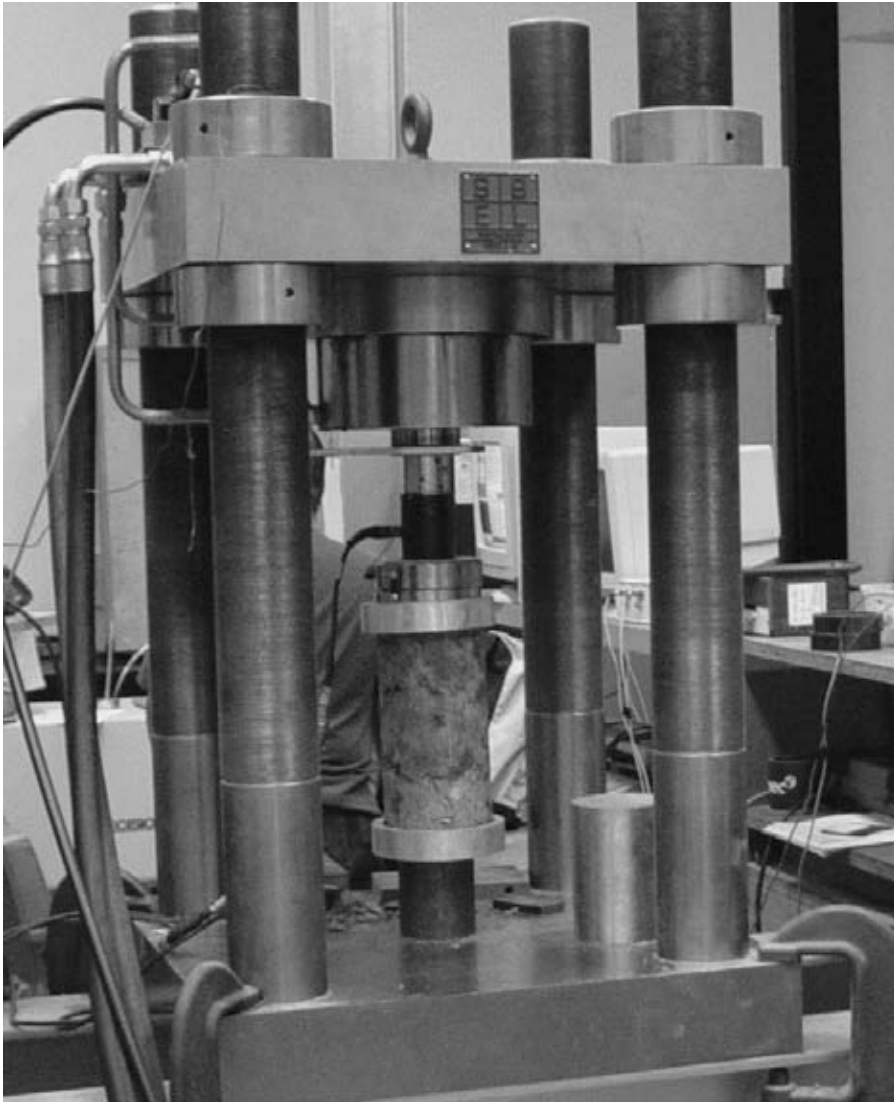


FIGURE 7.30 Compressive strength test.

than in small specimens. In general, large specimens have less variability and better representation of the actual strength of the concrete than small specimens. Therefore, the 0.15-m by 0.30-m (6-in. by 12-in.) size is the most suitable specimen size for determining the compressive strength. However, some agencies use 0.10-m (4-in.) diameter by 0.20-m (8-in.) high specimens. The advantages of using smaller specimens are the ease of handling, less possibility of accidental damage, less concrete needed, the ability to use a

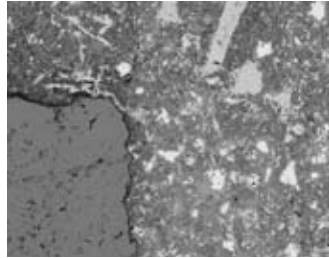


FIGURE 7.31 Scanning electron image showing the interface between a sand grain (lower left corner) and the paste.

low-capacity testing machine, and less space needed for curing and storage. Because of the strength variability of small specimens, more specimens should be tested for smaller specimens than are tested for standard-sized specimens. In some cases, five 0.10-m by 0.20-m replicate specimens are used instead of the three replicates commonly used for the standard-sized specimens. Also, when small-sized specimens are used, the engineer should understand the limitations of the test and consider these limitations in interpreting the results.

The interface between the hardened cement paste and aggregate particles is typically the weakest location within the concrete material. When concrete is stressed beyond the elastic range, microcracks develop at the cement paste–aggregate interface and continuously grow until failure. Figure 7.31 shows a scanning electron microscope micrograph of the fractured surface of a hardened cement mortar cylinder at 500x. The figure shows that the cleavage fracture surfaces where sand particles were dislodged during loading. The figure also shows the microcracks around some sand particles developed during loading.

7.5.2 ■ Split-Tension Test

The split-tension test (ASTM C496) measures the tensile strength of concrete. In this test a 0.15-m by 0.30-m (6-in. by 12-in.) concrete cylinder is subjected to a compressive load at a constant rate along the vertical diameter until failure, as shown in Figure 7.32. Failure of the specimen occurs along its vertical diameter, due to tension developed in the transverse direction. The split tensile (indirect tensile) strength is computed as

$$T = \frac{2P}{\pi Ld} \quad (7.4)$$

where

- T = tensile strength, MPa (psi),
- P = load at failure, N (psi),
- L = length of specimen, mm (in.), and
- d = diameter of specimen, mm (in.).

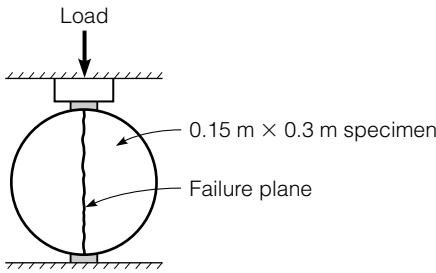


FIGURE 7.32 Split-tension test.

Typical indirect tensile strength of concrete varies from 2.5 MPa to 3.1 MPa (360 psi to 450 psi) (Neville 1981). The tensile strength of concrete is about 10% of its compressive strength.

7.5.3 Flexure Strength Test

The flexure strength test (ASTM C78) is important for design and construction of road and airport concrete pavements. The specimen is prepared either in the lab or in the field in accordance with ASTM C192 or C31, respectively. Several specimen sizes can be used. However, the sample must have a square cross section and a span of three times the specimen depth. Typical dimensions are 0.15-m by 0.15-m (6-in. by 6-in.) cross section and 0.30-m (18-in.) span. After molding, specimens are kept in the mold for the first 24 ± 8 hours, then removed from the mold and cured at $23 \pm 1.7^\circ\text{C}$ ($73.4 \pm 3^\circ\text{F}$), either in saturated-lime water or in a moist cabinet with a relative humidity of 95% or higher until testing. The specimen is then turned on its side and centered in the third-point loading apparatus, as illustrated in Figure 7.33. The load is continuously applied at a specified rate until rupture. If fracture initiates in the tension surface within the middle third of the span length, the flexure strength (modulus of rupture) is calculated as

$$R = \frac{Mc}{I} = \frac{PL}{bd^2} \tag{7.5}$$

where

- R = flexure strength, MPa (psi),
- M = maximum bending moment = $PL/6$, N.mm (lb.in.),
- c = $d/2$, mm (in.),
- I = moment of inertia = $bh^3/12$, mm^4 (in.^4),
- P = maximum applied load, which is distributed evenly (1/2 to each) over the two loading points, N (lb),
- L = span length, mm (in.),
- b = average width of specimen, mm (in.), and
- d = average depth of specimen, mm (in.).

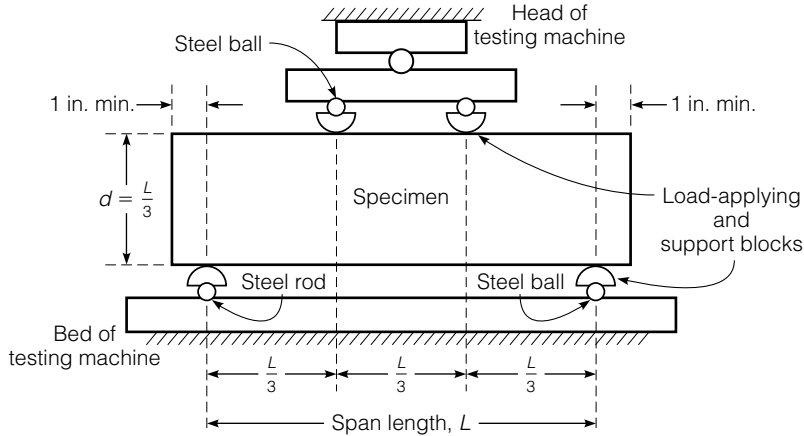


FIGURE 7.33 Apparatus for flexure test of concrete by third-point loading method (ASTM C78). Copyright ASTM. reprinted with permission.

Note that third-point loading ensures a constant bending moment without any shear force applied in the middle third of the specimen. Thus, Equation 7.5 is valid as long as fracture occurs in the middle third of the specimen. If fracture occurs slightly outside the middle third, the results can still be used with some corrections. Otherwise the results are discarded.

For normal-weight concrete, the flexure strength can be approximated as

$$R = (0.62 \text{ to } 0.83)\sqrt{f'_c} \quad (7.6a)$$

$$R = (7.5 \text{ to } 10)\sqrt{f'_c} \quad (7.6b)$$

Equation 7.6a is used for SI units, where both R and f'_c are in MPa, whereas Equation 7.6b is used for U.S. customary units, where both R and f' are in psi.

7.5.4 ■ Rebound Hammer Test

The rebound hammer test, also known as the Schmidt hammer test, is a non-destructive test performed on hardened concrete to determine the hardness of the surface (Figure 7.34). The hardness of the surface can be correlated, to some extent, with the concrete strength. The rebound hammer is commonly used to get an indication of the concrete strength. The device is about 0.3 m (1 ft) long and encloses a mass and a spring. The spring-loaded mass is released to hit the surface of the concrete. The mass rebounds, and the amount of rebound is read on a scale attached to the device. The larger the rebound, the harder is the concrete surface and, therefore, the greater is the strength. The device usually comes with graphs prepared by the manufacturer to relate



FIGURE 7.34 Rebound hammer for nondestructive evaluation of hardened concrete.

rebound to strength. The test can also be used to check uniformity of the concrete surface.

The test is very simple to run and is standardized by ASTM C805. To perform the test, the hammer must be perpendicular to a clean, smooth concrete surface. In some cases, it would be hard to satisfy this condition. Therefore, correlations, usually provided by the manufacturer, can be used to relate the strength to the amount of rebound at different angles. Rebound hammer results are also affected by several other factors, such as local vibrations, the existence of coarse aggregate particles at the surface, and the existence of voids near the surface. To reduce the effect of these factors, it is desirable to average 10 to 12 readings from different points in the test area.

7.5.5 ■ Penetration Resistance Test

The penetration resistance test, also known as the Windsor Probe test, is standardized by ASTM C803. The instrument (Figure 7.35) is a gunlike device that shoots probes into the concrete surface in order to determine its strength. The amount of penetration of the probe in the concrete is inversely related to the strength of concrete. The test is almost nondestructive since it creates small holes in the concrete surface.

The device is equipped with a special template with three holes, which is placed on the concrete surface. The test is performed in each of the holes. The average of the penetrations of the three probes through these holes is determined, using a scale and a special plate. Care should be exercised in handling the device to avoid injury. As a way of improving safety, the device cannot be operated without pushing hard on the concrete surface to prevent accidental shooting. The penetration resistance test is expected to provide better strength estimation than the rebound hammer, since the penetration resistance measurement is made not just at the surface but also in the depth of the sample.



FIGURE 7.35 Windsor probe test device.

7.5.6 ■ Ultrasonic Pulse Velocity Test

The ultrasonic pulse velocity test (ASTM C597) measures the velocity of an ultrasonic wave passing through the concrete (Figure 7.36). In this test, the path length between transducers is divided by the travel time to determine the average velocity of wave propagation. Attempts have been made to correlate pulse velocity data with concrete strength parameters. No good correlations were found, since the relationship between pulse velocity and strength data is affected by a number of variables, such as age of concrete, aggregate–cement ratio, aggregate type, moisture condition, and location of reinforcement (Mehta and Moneiro 1993). This test is used to detect cracks, discontinuities, or internal deterioration in the structure of concrete.

7.5.7 ■ Maturity Test

Maturity of a concrete mixture is defined as the degree of cement hydration, which varies as a function of both time and temperature. Therefore, it is assumed that, for a particular concrete mixture, strength is a function of maturity. Maturity meters (Figure 7.37) have been developed to provide an estimate of concrete strength by monitoring the temperature of concrete with time. This test (ASTM C1074) is performed on fresh concrete and continued for several days. The maturity meter must be calibrated for each concrete mix.



FIGURE 7.36 Ultrasonic pulse velocity apparatus (Courtesy of James Instrument Inc.).



FIGURE 7.37 Checking the maturity of fresh concrete using the maturity meter.

7.6 Alternatives to Conventional Concrete

There are several alternatives that increase the flexibility and applications of concrete. While a technical presentation of materials for each of these technologies is beyond the scope of this book, the engineer should be aware of some of the materials used to provide additional capabilities of concrete. Some of these alternatives include the following:

- self-consolidating concrete
- flowable fill
- Shotcrete
- lightweight concrete
- high-strength concrete
- shrinkage-compensating concrete
- fiber-reinforced concrete
- heavyweight concrete
- polymers and concrete
- high-performance concrete roller-compacted concrete

7.6.1 Self-Consolidating Concrete

Self-consolidating concrete (SCC), also known as self-compacting concrete, is a highly flowable, nonsegregating concrete that can spread into place, fill the formwork, and encapsulate the reinforcement, without any mechanical consolidation (NRMCA). Some of the advantages of using SCC are the following:

1. It can be placed at a faster rate, with no mechanical vibration and less screeding, resulting in savings in placement costs.
2. It improves and makes more uniform the architectural surface finish with little or no remedial surface work.
3. It improves ease of filling restricted sections and hard-to-reach areas. This allows the designer to create structural and architectural shapes not achievable with conventional concrete.
4. It improves consolidation around reinforcement and bond with reinforcement.
5. It improves pumpability.

Two important properties specific to self-consolidating concrete in its plastic state are its *flowability* and *stability*. The high flowability of SCC is achieved by using high-range water-reducing admixtures without adding extra mixing water. The stability, or resistance to segregation, is attained by increasing the amount of fines and/or by using admixtures that modify the viscosity of the mixture. Fines could be either cementitious materials or mineral fines.

The most common field test that has been used to measure the flowability and stability of SCC is a modified version of the slump test, discussed



FIGURE 7.38 Measuring the spread during the slump flow test.

earlier. In this slump flow test, the slump cone is completely filled, without consolidation. The cone is lifted, and the spread of the concrete is measured, as shown in Figure 7.38. The flowability is measured by the spread, or slump flow. The spread typically ranges from 455 to 810 mm (18 to 32 inches), depending on the requirements of the project. The resistance to segregation, or stability, is measured with the visual stability index (VSI). The VSI is established on the basis of whether bleed water is observed at the leading edge of the spreading concrete or aggregates pile at the center. VSI values range from 0 for “highly stable” to 3 for unacceptable stability.

During the slump flow test, the viscosity can be measured by the rate at which the concrete spreads. The time taken for the concrete to reach a spread diameter of 50 cm (20 inches) from the moment the slump cone is lifted up is measured. This is called the T_{50} (T_{20}) measurement and typically varies between 2 and 10 seconds for SCC. A T_{50} (T_{20}) value indicates a more viscous mix, which is more appropriate for concrete in applications with congested reinforcement or in deep sections. A lower T_{50} (T_{20}) value may be appropriate for concrete that has to travel long horizontal distances without much obstruction.

7.6.2 ■ Flowable Fill

Flowable fill is a self-leveling and self-compacting, cementitious material with an unconfined compressive strength of 8.3 MPa (1200 psi) or less. Flowable fill is primarily used as a backfill material in lieu of compacted granular fill (Figure 7.39). Flowable fill is also commonly referred to as controlled low-strength material (CLSM), controlled density fill (CDF), flowable compacting fill, lean fill, unshrinkable fill, flow mortar, fly ash flow, and liquid dirt (NRMCA).



FIGURE 7.39 Filling a sink hole with flowable fill.

The flowable fill mix consists of cement, sand, and water typically mixed with fly ash, ground granulated blast furnace (GGBF) slag, and/or air generating admixtures. The type and proportions of ingredients used to make the flowable fill can largely change its properties to match its intended use.

One of the unique properties of flowable fill that makes it advantageous compared with compacted granular fill is its flowability. High flowability and self-leveling characteristics allow flowable fill to eliminate voids and access spaces that prove to be difficult or impossible with compacted granular fill.

When the flowable fill is placed in the cavity, its initial in-place volume is slightly reduced in the order of 10 mm per meter (1/8 inch per foot) of depth. This settlement is caused by the displacement of water and the release of entrapped air as a result of consolidation. Once the flowable fill is hardened, the settlement stops and the volume does not change.

Flowable fill can be proportioned to have a wide range of compressive strength. However, the most commonly used flowable fill mixtures are proportioned with consideration of possible excavation in future years and range in compressive strengths between 0.35 MPa and 1 MPa (50 and 150 psi). Flowable fill with a compressive strength less than 150 psi may be excavated manually, while at the same time having adequate bearing capacity to support external loads, such as the weight of a vehicle. Flowable fill exhibiting strengths between 150 and 300 psi will require mechanical equipment for excavation. If the strength exceeds 2 MPa (300 psi), the material is not considered excavatable.

Flowable fill has several advantages over compacted granular backfill. Flowable fill does not require compaction, which is a main concern with granular backfill. Flowable fill can also reach inaccessible locations, such as places around pipes, which are hard to reach with granular backfill. Flowable fill also has a greater bearing capacity than compacted granular fill and no noticeable settlement. It can even be placed in standing water. These advantages result in reduced in-place costs for labor and equipment, as well as time saving during construction.

Flowable fill is typically used as backfill for utility trenches, retaining walls, pipe bedding, and building excavation. It is also used as structural fill for poor soil conditions, mud jacking, floor and footing support, and bridge conversion. Other uses include pavement base, erosion control, and void filling. Flowable fill has been getting more common in recent years and is mostly available at local ready-mix producers.

7.6.3 ■ Shotcrete

Shotcrete is mortar or small-aggregate concrete that is sprayed at high velocity onto a surface (see Figures 7.40 and 7.41). Shotcrete, also known as “gunite” and “sprayed concrete,” is a relatively dry mixture that is consolidated by the impact force and can be placed on vertical or horizontal surfaces without sagging.



FIGURE 7.40 Constructing a swimming pool using shotcrete.



FIGURE 7.41 Shotcrete used in lining a tunnel.

Shotcrete is applied by either the dry or wet process. In the dry process, a premixed blend of cement and damp aggregate is propelled through a hose by compressed air to a nozzle, while the water is added at the nozzle. In the wet process, all ingredients are premixed and pumped through a hose to the nozzle and forced to the surface using compressed air. In either case, the nozzle should be held perpendicular to the surface to reduce the rebound of coarse aggregates off the surface. The nozzle is held about 0.5 to 1.5 m (1.5 to 5 ft) away from the surface.

Supplementary cementitious materials, such as fly ash and silica fume, can be used in shotcrete to improve workability, chemical resistance, and durability. Accelerating admixtures can also be used to reduce the time of initial set and to allow buildup of thicker layers of shotcrete in a single pass. Steel fibers may also be used to improve flexural strength, ductility, and toughness (Kosmatka et al. 2002).

7.6.4 ■ Lightweight Concrete

Students competing in the annual ASCE concrete canoe competition frequently produce concrete with a unit weight less than water. The *ACI Guide*

for the *Structural Lightweight Aggregate Concrete* requires a 28-day compressive strength of 17 MPa (2500 psi) and an air-dried unit weight of less than 1850 kg/m^3 (115 lb/ft^3) for structural lightweight concrete. The use of lightweight concrete in a structure is usually predicated on the overall cost of the structure; the concrete may cost more, but the reduced dead weight can reduce structural and foundation costs.

The mix proportions for lightweight concrete must compensate for the absorptive nature of the aggregates. Generally, lightweight aggregates are highly absorptive and can continue to absorb water for an extended period of time (Figures 7.42 and 7.43). This makes the determination of a water–cementitious materials ratio problematical. In addition, the lightweight aggregates tend to segregate by floating to the surface. A minimum slump mix, with air entraining, is used to mitigate this effect.

Nonstructural applications of very lightweight concrete have also been developed. Concrete made with styrofoam “aggregates” has been used for insulation in some building construction.

7.6.5 ■ Heavyweight Concrete

Biological shielding used for nuclear power plants, medical units, and atomic research and test facilities requires massive walls to contain radiation. Concrete is an excellent shielding material. For biological shields, the mass of



FIGURE 7.42 Aggregate used for lightweight concrete.



FIGURE 7.43 ASCE students at Arizona State University making a concrete canoe using lightweight concrete.

the concrete can be increased with the use of heavyweight aggregates. The aggregates can be either natural or manufactured. Natural heavyweight aggregates include barite, magnetite, hematite, geothite, illmenite, and ferrophosphorus. The specific gravity of these aggregates range from 3.4 to 6.5. Steel, with a specific gravity of 7.8, can be used as aggregate for heavyweight concrete (Figure 7.44). However, the specific gravity of the aggregates makes workability and segregation of heavyweight concrete problematical. Using a higher proportion of sand improves the workability. The workability problem can be avoided by preplacing aggregate, then filling the voids between aggregate particles with grout of cement, sand, water, and admixtures. ASTM C637, *Specifications for Aggregates for Radiation Shielding Concrete* and ASTM C637, *Nomenclature of Constituents of Aggregates for Radiation Shielding Concrete* provide further information on heavyweight concrete practices.

7.6.6 ■ High-Strength Concrete

Concrete made with normal-weight aggregate and having compressive strengths greater than 40 MPa (6000 psi) is considered to be high-strength concrete. Producing a concrete with more than 40 MPa compressive strength requires care in the proportioning of the components and in quality control



FIGURE 7.44 Steel used as aggregate for heavy weight concrete.

during construction. The microstructure of concrete with a compressive strength greater than 40 MPa is considerably different from that of conventional concrete. In particular, the porosity of the cement paste and the transition zone between the cement paste and the aggregate are the controlling factors for developing high strength. This porosity is controlled by the water–cementitious materials ratio. The development of superplasticizers has permitted the development of high-strength concrete that is workable and flowable at low water–cementitious materials ratios. In addition, high-strength concrete has excellent durability due to its tight pore structure. In the United States, high-strength concrete is used primarily for skyscrapers. The high-strength and corresponding high elastic modulus allow for reduced structural element size.

7.6.7 ■ Shrinkage-Compensating Concrete

Normal concrete shrinks at early ages, especially if it is not properly cured, as discussed earlier in this chapter. The addition of alumina powders to the cement can cause the concrete to expand at early ages. Shrinkage-compensating cement is marketed as Type K cement. Expansive properties can be used to advantage by restraining the concrete, either by reinforcing or by other means, at early ages. As the restrained concrete tries to expand, compressive stresses are developed. These compressive stresses reduce the tensile stresses developed by drying shrinkage, and the chance of the concrete cracking due to drying shrinkage is reduced. Details on the design and use of shrinkage-compensating concrete are available in ACI Committee 223 report *Recommendations for the Use of Shrinkage-Compensating Cements*.

7.6.8 ■ Polymers and Concrete

Polymers can be used in several ways in the production of concrete. The polymer can be used as the sole binding agent to produce *polymer concrete*. Polymers can be mixed with the plastic concrete to produce *polymer–portland cement concrete*. Polymers can be applied to hardened concrete to produce *polymer-impregnated concrete*.

Polymer concrete is a mixture of aggregates and a polymer binder. There are a wide variety of polymers that can be mixed with aggregates to make polymer concrete. Some of these can be used to make rapid-setting concrete that can be put in service in under an hour after placement. Others are formulated for high strength; 140 MPa (20,000 psi) is possible. Some have good resistance to chemical attack. A common characteristic is that most polymer concretes are expensive, limiting their application to situations in which their unique characteristics make polymer concrete a cost-effective alternative to conventional concrete.

Polymer–portland cement concrete incorporates a polymer into the production of the portland cement concrete. The polymer is generally an elastomeric emulsion, such as latex.

7.6.9 ■ Fiber-Reinforced Concrete

The brittle nature of concrete is due to the rapid propagation of microcracking under applied stress. However, with fiber-reinforced concrete, failure takes place due to fiber pull-out or debonding. Unlike plain concrete, fiber-reinforced concrete can sustain load after initial cracking. This effectively improves the toughness of the material. In addition, the flexural strength of the concrete is improved by up to 30%. For further information on the design and applications of fiber-reinforced concrete, consult the *ACI Guide for Specifying, Mixing, Placing and Finishing Steel Fiber Reinforced Concrete*.

Fibers are available in a variety of sizes, shapes, and materials (Figure 7.45). The fibers can be made of steel, plastic, glass, and natural materials. Steel fibers are the most common. The shape of fibers is generally described by the aspect ratio, length/diameter. Steel fibers generally have diameters from 0.25 mm to 0.9 mm (0.01 in. to 0.035 in.) with aspect ratios of 30 to 150. Glass fiber elements' diameters range from 0.013 mm to 1.3 mm (0.005 in. to 0.05 in.).



FIGURE 7.45 Fibers used for fiber-reinforced concrete

The addition of fibers to concrete reduces the workability. The extent of reduction depends on the aspect ratio of the fibers and the volume concentration. Generally, due to construction problems, fibers are limited to a maximum of 2% by volume of the mix. Admixtures can be used to restore some of the workability to the mix.

Since the addition of fibers does not greatly increase the strength of concrete, its use in structural members is limited. In beams, columns, suspended floors, etc., conventional reinforcing must be used to carry the total tensile load. Fiber-reinforced concrete has been successfully used for floor slabs, pavements, slope stabilization, and tunnel linings.

7.6.10 ■ Roller-Compacted Concrete

Based on the unique requirements for mass concrete used for dam construction, roller-compacted concrete (RCC) was developed. This material uses a relatively low cement factor, relaxed gradation requirements, and a water content selected for construction considerations rather than strength. RCC is a no-slump concrete that is transported, placed, and compacted with equipment used for earth and rockfill dam construction. The RCC is hauled by dump trucks, spread with bulldozers, and compacted with vibration compactors. Japanese experience using RCC in construction found several advantages:

1. The mix is economical, because of the low cement content.
2. Formwork is minimal, because of the layer construction method.
3. The low cement factor limits the heat of hydration, reducing the need for external cooling of the structure.
4. The placement costs are lower than those for conventional concrete methods, due to the use of high-capacity equipment and rapid placement rates.
5. The construction period is shorter than that for conventional concrete.

In addition, experience in the United States has demonstrated that RCC in-place material costs are about one-third those of conventional concrete. The two primary applications of RCC have been for the construction of dams and large paved areas, such as military tank parking aprons.

7.6.11 ■ High-Performance Concrete

While the current specifications for concrete have provided a material that performs reasonably well, there is concern that the emphasis on strength in the mix design process has led to concrete that is inadequate in other performance characteristics. This has led to an interest in developing specifications and design methods for what has been termed *high-performance concrete* (HPC). The American Concrete Institute (ACI) defines HPC as concrete that meets special performance and uniformity requirements, which cannot always be obtained using conventional ingredients, normal mixing

procedures, and typical curing practices. These requirements may include the following enhancements:

- ease of placement and compaction
- long-term mechanical properties
- early-age strength
- toughness
- volume stability
- extended life in severe environments

These enhanced characteristics may be accomplished by altering the aggregate gradation, including special admixtures, and improving mixing and placement practices. Currently, a compressive strength in the order of 70 to 175 MPa (10,000 to 25,000 psi) can be obtained. As the need for HPC is better understood and embraced by the engineering community, there will probably be a transition in concrete specification from the current prescriptive method to the performance-based or performance-related specifications. A Strategic Highway Research Program (SHRP) study (Zia et al. 1991) defined HPC as

- maximum water–cementitious materials ratio of 0.35;
- minimum durability factor of 80%, as determined by ASTM C 666, Procedure A; and
- minimum strength criteria of either:
 - a. 21 MPa (3000 psi) within 4 hours after placement (Very Early Strength, VES),
 - b. 34 MPa (5000 psi) within 24 hours (High Early Strength, HES), or
 - c. 69 MPa (10,000 psi) within 28 days (Very High Strength, VHS).

Thus, high-performance concrete is characterized by special performance, both short-term and long-term, and uniformity in behavior (Nawy 1996). Such requirements cannot always be achieved by using only conventional materials or applying conventional practices. Since concrete is the most widely used construction material worldwide, new concrete construction has to utilize the currently available new technology in order to eliminate costly future rehabilitation.

Revolutionary new construction materials and modifications and improvements in the behavior of traditional materials have been developed in the last three decades. These developments have been considerably facilitated by increased knowledge of the molecular structure of materials, studies of long-term failures, development of more powerful instrumentation and monitoring techniques, and the need for stronger and better performing materials, suitable for larger structures, longer spans, and more ductility.

In spite of the current advances of the concrete technology and the development of high-performance concretes, it is expected that the concrete industry will continue improving through the development of new components and admixtures, microstructural studies, blended cement compositions, better material selection and proportioning techniques, and more efficient placement techniques.

S U M M A R Y

The design of durable portland cement concrete materials is the direct responsibility of civil engineers. Selection of the proper proportions of portland cement, water, aggregates, and admixtures, along with good construction practices, dictates the quality of concrete used in structural applications. Using the volumetric mix design method presented in this chapter will lead to concrete with the required strength and durability. However, the proper design of portland cement concrete is irrelevant unless proper construction procedures are followed, including the appropriate mixing, transporting, placing, and curing of the concrete. To ensure that these processes produce concrete with the desired properties, a variety of quality control tests are performed by civil engineers, including slump tests, air content tests, and strength-gain-with-time tests.

While the vast majority of concrete projects are constructed with conventional materials, there are a variety of important alternative concrete formulations available for specialty applications. These alternatives are introduced in this chapter; however, the technology associated with these alternatives is relatively complex, and further study is required in order to fully understand the behavior of these materials.

Q U E S T I O N S A N D P R O B L E M S

- 7.1 The design engineer specifies a concrete strength of 5500 psi. Determine the required average compressive strength for
 - a. a new plant where s is unknown
 - b. a plant where $s = 500$ psi for 22 test results
 - c. a plant with extensive history of producing concrete with $s = 400$ psi
 - d. a plant with extensive history of producing concrete with $s = 600$ psi
- 7.2 A project specifies a concrete strength of 24.1 MPa. Materials engineers will design the mix for a strength higher than that to account for variabilities.
 - a. Calculate the required average compressive strength of the mix design if the mixing plant has a standard deviation of $s = 3.8$ MPa.
 - b. Using the ACI code equation, estimate what would be the modulus of elasticity of this concrete at the *required* compressive strength.

- 7.3 A project specifies a concrete strength of at least 3000 psi. Materials engineers will design the mix for a strength higher than that. Calculate the required average compressive strength of the mix design if the standard deviation is $s = 350$ psi. Estimate the modulus of elasticity of the concrete at the required average compressive strength (the calculated strength, not the given strength).
- 7.4 What is your recommendation for the maximum nominal size of coarse aggregate for the following situation?
A continuously reinforced concrete pavement cross section contains a layer of No. 6 reinforcing bars at 6-inch centers, such that the steel is just above mid-depth of a 10-inch thick slab. Cover over the top of the steel is therefore about 4 inches.
- 7.5 A concrete mix with a 3-inch slump, w/c ratio of 0.50, and sand with a fineness modulus of 2.4 contains 1700 lb of coarse aggregate. Compute the required weight of coarse aggregate per cubic yard. To adjust the mix so as to increase the compressive strength, the water-to-cement ratio is reduced to 0.45. Will the quantity of coarse aggregate increase, decrease, or stay the same? Explain your answer.
- 7.6 You are working on a concrete mix design that requires each cubic yard of concrete to have 0.45 water-to-cementitious materials ratio, 1963 lb/yd³ of dry gravel, 4% air content, and 565 lb/ft³ of cement. The available gravel has a specific gravity of $G_{\text{gravel}} = 2.7$, a moisture content of 1.6%, and absorption of 2.4%. The available sand has a specific gravity of $G_{\text{sand}} = 2.5$, a moisture content of 4.8%, and absorption of 1.5%. For each cubic yard of concrete needed on the job, calculate the weights of cement, moist gravel, moist sand, and water that should be added to the batch. Summarize and total the mix design when finished (don't include air entrainer in summary).
- 7.7 Design the concrete mix according to the following conditions:

Design Environment

Building frame

Required design strength = 27.6 MPa

Minimum dimension = 150 mm

Minimum space between rebar = 40 mm

Minimum cover over rebar = 40 mm

Statistical data indicate a standard deviation of compressive strength of 2.1 MPa is expected (more than 30 samples).

Only air entrainer is allowed.

Available Materials

Air entrainer: Manufacture specification 6.3 ml/1% air/100 kg cement.

Coarse aggregate: 19 mm maximum size, river gravel (rounded)

Bulk oven-dry specific gravity = 2.55, Absorption = 0.3%

Oven-dry rodded density = 1761 kg/m³

Moisture content = 2.5%

Fine aggregate: Natural sand

Bulk oven-dry specific gravity = 2.659, Absorption = 0.5%
 Moisture content = 2%
 Fineness modulus = 2.47

- 7.8. Design the concrete mix according to the following conditions.

Design Environment

Pavement slab, Bozeman, Montana (cold climate)

Required design strength = 3000 psi

Slab thickness = 12 in.

Statistical data indicate a standard deviation of compressive strength of 250 psi is expected (more than 30 samples).

Only air entrainer is allowed.

Available Materials

Air entrainer: Manufacture specification is 0.15 fl oz/1% air/100 lb cement.

Coarse aggregate: 2 in. maximum size, crushed gravel (angular)

Bulk oven-dry specific gravity = 2.573, Absorption = 0.1%

Oven-dry rodded density = 120 pcf

Moisture content = 1%

Fine aggregate: Natural sand

Bulk oven-dry specific gravity = 2.54, Absorption = 0.2%

Moisture content = 3.67%

Fineness modulus = 2.68

- 7.9 The design of a concrete mix requires 1173 kg/m^3 of gravel in dry condition, 582 kg/m^3 of sand in dry condition, and 157 kg/m^3 of free water. The gravel available at the job site has a moisture content of 0.8% and absorption of 1.5%, and the available sand has a moisture content of 1.1% and absorption of 1.3%. What are the masses of gravel, sand, and water per cubic meter that should be used at the job site?
- 7.10 Design a non-air-entrained concrete mix for a small job with a maximum gravel size of 25 mm (1 in.). Show the results as follows:
- masses of components to produce 2000 kg (4400 lb) of concrete.
 - volumes of components to produce 1 m^3 (36 ft^3) of concrete.
- 7.11 Design a non-air-entrained concrete mix for a small job with a maximum gravel size of $\frac{3}{4}$ in. Show the results as follows:
- weights of components to produce 5000 lb of concrete.
 - volumes of components to produce 1 yd^3 of concrete.
- 7.12 Why is it necessary to measure the air content of concrete at the job site rather than at the batch plant? Name one of the methods used to measure the air content of concrete.
- 7.13 What do we mean by curing concrete? What would happen if concrete is not cured?
- 7.14 Discuss five different methods of concrete curing.
- 7.15 Discuss the change in volume of concrete at early ages.

- 7.16 Discuss the creep response of concrete structures. Provide examples of the effect of creep on concrete structures.
- 7.17. On one graph, draw a sketch showing the typical relationship between the stress and strain of concrete specimens with high and low water–cementitious materials ratios. Label all axes and curves. Comment on the effect of increasing the water–cementitious materials ratio on the stress–strain response.
- 7.18 Using Figure 7.29,
- Determine the ultimate stress at each water–cementitious materials ratio.
 - Determine the secant modulus at 40% of the ultimate stress at each water–cementitious materials ratio.
 - Plot the relationship between the secant moduli and the ultimate stresses.
 - Plot the relationship between the moduli and the ultimate stresses on the same graph of part (c), using the relation of the ACI Building Code (Equation 7.3).
 - Compare the two relations and comment on any discrepancies.
- 7.19 A normal-weight concrete has an average compressive strength of 4500 psi. What is the estimated modulus of elasticity?
- 7.20 Discuss the significance of the compressive strength test on concrete. Draw a graph to show the relationship between compressive strength and water–cementitious materials ratio for different curing times (Label all axes and curves).
- 7.21 What is the standard size of PCC specimens to be tested for compressive strength? If a smaller size is used, which size is expected to provide higher compressive strengths? Why? Which size provides strengths close to that of an actual concrete structure?
- 7.22 A short plain concrete column with dimensions of 12 in. \times 12 in. \times 36 in. is to be constructed. If the compressive strength of the concrete is 5000 psi, what is the maximum load that can be applied to this column using a factor of safety of 1.2?
- 7.23 What is the purpose of performing the flexure test on concrete? How are the results of this test related to the compressive strength test results?
- 7.24 What are the advantages of using a third-point loading flexure test over a center-point loading flexure test? Draw a shear force diagram and a bending moment diagram for each case to support your answer.
- 7.25 Consider a standard flexural strength specimen of length L , width a , and depth a . Assume third point loading where the load at failure from the test machine is P , which is distributed evenly (1/2 to each) over the two loading points. Derive the equation for calculating the modulus of rupture of the beam in terms of P , L , and a .

- 7.26 The flexure strength test was performed on a concrete beam having a cross section of 0.15 m by 0.15 m and a span of 0.45 m. If the load at failure was 35.7 kN, calculate the flexure strength of the concrete.
- 7.27 A normal-weight concrete has an average compressive strength of 20 MPa. What is the estimated flexure strength?
- 7.28 Discuss two nondestructive tests to be performed on hardened concrete. Show the basic principles behind the tests and how they are performed.
- 7.29 Discuss the concept of concrete maturity meters.
- 7.30 Discuss four alternatives that increase the use and application of conventional concrete.
- 7.31 What is self-consolidating concrete? How are its properties achieved? How are these properties measured?
- 7.32 What is flowable fill? Discuss its ingredients and its advantages.
- 7.33 Discuss the concept of high-performance concrete. Discuss some of its properties that make it preferred over conventional concrete.
- 7.34 Comparing PCC with mild steel, answer the following questions:
 - a. Which one is stronger?
 - b. Which one has a higher modulus or stiffness?
 - c. Which one is more brittle?
 - d. What is the range of compressive strength for a typical PCC?
 - e. What is the compressive strength for a high-strength concrete?
 - f. What would be a reasonable range for PCC modulus?
- 7.35 In a ready-mix plant, cylindrical samples are prepared and tested periodically to detect any mix problem and to ensure that the compressive strength is higher than the lower specification limit. The minimum target value was set at 4000 psi. The following compressive strength data were collected.

Sample No.	Compressive Strength (psi)	Sample No.	Compressive Strength (psi)
1	4914	14	5772
2	4732	15	4270
3	5670	16	5096
4	4310	17	4670
5	6110	18	5174
6	4316	19	5434
7	5240	20	3692
8	4950	21	4510
9	5230	22	3680
10	4190	23	4100
11	5770	24	3680
12	4524	25	3910
13	4056		

- a. Calculate the mean, standard deviation, and the coefficient of variation of the data.
- b. Using a spreadsheet program, create a control chart for these data showing the target value and the lower specification limit. Is the plant production meeting the specification requirement? If not, comment on possible reasons. Comment on the data scatter.

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8

MASONRY



A masonry structure is formed by combining masonry units, such as stone or brick, with mortar. Masonry is one of the oldest construction materials. Examples of ancient masonry structures include the pyramids of Egypt, the Great Wall of China, and Greek and Roman ruins. Bricks of nearly uniform size became commonly used in Europe during the beginning of the 13th century. The first extensive use of bricks in the United States was around 1600. In the last two centuries, bricks have been used in constructing sewers, bridge piers, tunnel linings, and multistory buildings. Masonry units (Figure 8.1) are still being used in construction in the United States and are competing with other materials, such as wood, steel, and concrete (Adams 1979).

8.1 Masonry Units

Masonry units can be classified as

- concrete masonry units
- clay bricks
- structural clay tiles
- glass blocks
- stone

Concrete masonry units can be either solid or hollow, but clay bricks, glass blocks, and stone are typically solid. Structural clay tiles are hollow units that are larger than clay bricks and are used for lightweight masonry, such as partition walls and filler panels. They can be used with their webs in either a horizontal or a vertical direction. Figure 8.2 shows examples of concrete masonry units, clay bricks, and structural clay tiles. Concrete masonry units and clay bricks are commonly used in the United States.



FIGURE 8.1 Masonry units used in construction.

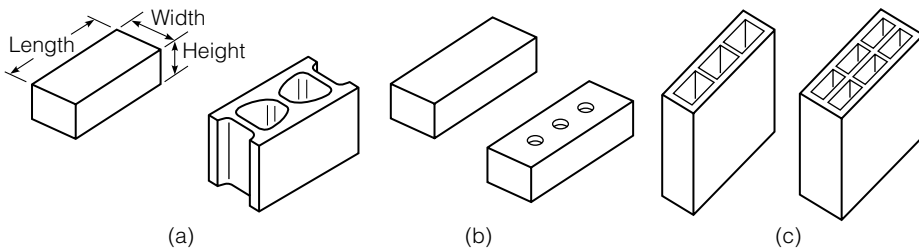


FIGURE 8.2 Examples of masonry units: (a) concrete masonry units, (b) clay bricks, and (c) structural clay tiles

8.1.1 ■ Concrete Masonry Units

Solid concrete units are commonly called concrete bricks, whereas hollow units are known as concrete blocks, hollow blocks, or cinder blocks. Hollow units have net cross-sectional area in every plane parallel to the bearing surface with less than 75% of the cross-sectional area in the same plane. If this ratio is 75% or more, the unit is categorized as solid (Portland Cement Association 1991).

Concrete masonry units are manufactured in three classes, based on their density: *lightweight units*, *medium-weight units*, and *normal-weight units*,

TABLE 8.1 Weight Classifications and Allowable Maximum Water Absorption of Concrete Masonry Units (ASTM C90 and C129) (Copyright ASTM, reprinted with permission).

Weight Classification	Unit Weight Mg/m³ (pcf)	Maximum Water Absorption kg/m³ (lb/ft³) (Average of 3 units)
Lightweight	1.68 (105)	288 (18)
Medium Weight	1.68–2.00 (105–125)	240 (15)
Normal Weight	2.00 (125) or more	208 (13)

with dry unit weights as shown in Table 8.1. Well-graded sand, gravel, and crushed stone are used to manufacture normal-weight units. Lightweight aggregates such as pumice, scoria, cinders, expanded clay, and expanded shale are used to manufacture lightweight units. Lightweight units are the most common concrete units used in masonry construction because they are easy to handle and transport, and the weight of the structure is reduced. Lightweight units have higher thermal and fire resistance properties and lower sound resistance than normal weight units.

Concrete masonry units are manufactured using a relatively dry (zero-slump) concrete mixture consisting of portland cement, aggregates, water, and admixtures. Type I cement is usually used to manufacture concrete masonry units; however, Type III is sometimes used to reduce the curing time. Air-entrained concrete is sometimes used to increase the resistance of the masonry structure to freeze and thaw effects and to improve workability, compaction, and molding characteristics of the units during manufacturing. The units are molded under pressure, then cured, usually using low-pressure steam curing. After manufacturing, the units are stored under controlled conditions so that the concrete continues curing.

Concrete masonry units can be classified as load bearing (ASTM C90) and non-load bearing (ASTM C129). Load-bearing units must satisfy a higher minimum compressive strength requirement than non-load-bearing units, as shown in Table 8.2. The compressive strength of individual concrete masonry units is determined by capping the unit and applying load in the direction of the height of the unit until failure (ASTM C140). A full-size unit is recommended for testing, although a portion of a unit can be used if the capacity of the testing machine is not large enough. The *gross area compressive strength* is calculated by dividing the load at failure by the gross cross-sectional area of the unit. The *net area compressive strength* is calculated by dividing the load at failure by the net cross-sectional area. The net cross-sectional area is calculated by dividing the net volume of the unit by its average height. The net volume is determined using the water displacement method according to ASTM C140.

TABLE 8.2 Strength Requirements of Concrete Masonry Units (ASTM C90 and C129)

Type	Minimum Compressive Strength Based on Net Area MPa (psi)	
	Average of Three Units	Individual Units
Load bearing	13.1 (1900)	11.7 (1700)
Non-load-bearing	4.1 (600)	3.5 (500)

Sample Problem 8.1

A hollow concrete masonry unit has actual gross dimensions of 7-5/8 in. \times 7-5/8 in. \times 15-5/8 in. The unit is tested in a compression machine with the following results:

Failure Load = 250 kips

Net volume of 366.9 in³

- Calculate the gross area compressive strength.
- Calculate the net area compressive strength.

Solution

- Gross area = 7.625 \times 15.625 = 119.141 in²
Gross area compressive strength = 250,000/119.141 = 2,098.4 psi
- Net area = 366.9/7.625 = 48.118 in²
Net area compressive strength = 250,000/48.118 = 5,195.6 psi

The amount of water absorption of concrete masonry units is controlled by ASTM standards to reduce the effect of weathering and to limit the amount of shrinkage due to moisture loss after construction (ASTM C90). The absorption of concrete masonry units is determined by immersing the unit in water for 24 hours (ASTM C140). The absorption and moisture content are calculated as follows.

$$\text{Absorption (kg/m}^3\text{)} = \frac{W_s - W_d}{W_s - W_i} \times 1000 \quad (8.1)$$

$$\text{Absorption (lb/ft}^3\text{)} = \frac{W_s - W_d}{W_s - W_i} \times 62.4 \quad (8.2)$$

$$\text{Absorption}(\%) = \frac{W_s - W_d}{W_d} \times 100 \quad (8.3)$$

$$\text{Moisture content as a percent of total absorption} = \frac{W_r - W_d}{W_s - W_d} \times 100 \quad (8.4)$$

where

W_s = saturated weight of specimen, kg (lb)

W_d = oven-dry weight of unit, kg (lb),

W_i = immersed weight of specimen, kg (lb), and

W_r = weight of specimen as received

Table 8.1 shows the allowable maximum water absorption for load-bearing concrete masonry units.

Sample Problem 8.2

A concrete masonry unit was tested according to ASTM C140 procedure and produced the following results:

mass of unit as received = 10,354 g

saturated mass of unit = 11,089 g

oven-dry mass of unit = 9893 g

Calculate the percent absorption and moisture content of the unit as a percent of total absorption.

Solution

$$\text{Absorption}(\%) = \frac{11,089 - 9893}{9893} \times 100 = 12.1\%$$

Moisture content as a percent of total absorption

$$= \frac{(10,354 - 9893)}{(11,089 - 9893)} \times 100 = 38.5\%$$

Concrete masonry units are available in different sizes, colors, shapes, and textures. Concrete masonry units are specified by their nominal dimensions. The *nominal dimension* is greater than its *specified* (or *modular*) dimension by the thickness of the mortar joint, usually 10 mm (3/8 in.). For example, a 200 mm × 200 mm × 400 mm (8 in. × 8 in. × 16 in.) block has an actual width of 190 mm (7-5/8 in.), height of 190 mm (7-5/8 in.), and length of 390 mm (15-5/8 in.), as illustrated in Figure 8.3. Load-bearing concrete

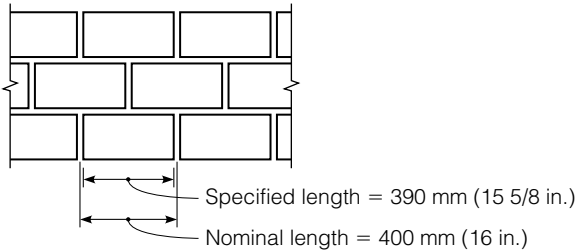


FIGURE 8.3 Nominal dimensions and specified (modular) dimensions.

masonry units are available in nominal widths of 100 mm, 150 mm, 200 mm, 250 mm, and 300 mm (4 in., 6 in., 8 in., 10 in., and 12 in.), heights of 100 mm and 200 mm (4 in. and 8 in.), and lengths of 300 mm, 400 mm, and 600 mm (12 in., 16 in., and 24 in.). Common load-bearing blocks are 200 mm × 200 mm × 400 mm, whereas non-load-bearing blocks are 100 mm × 200 mm × 400 mm, as shown in Figure 8.4. Also, depending on the position within the masonry wall, they are manufactured as stretcher, single-corner, and double-corner units, as depicted in Figure 8.5.

Solid concrete masonry units (concrete bricks) are manufactured in two grades (N and S) based on strength and absorption requirements. Grade N units have higher compressive strength, resistance to moisture penetration, and resistance to frost action than grade S. According to ASTM C55, the minimum compressive strength of individual units is 20.7 MPa (3000 psi) for grade N, and 13.8 MPa (2000 psi) for grade S. Grade N bricks are typically used as architectural veneers and facing units in exterior walls. Grade S bricks are for general use where moderate strength and resistance to frost action and moisture penetration are required.

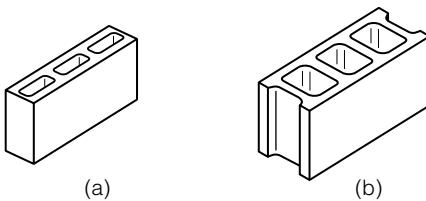


FIGURE 8.4 Concrete masonry units: (a) Non-load-bearing and (b) load-bearing.

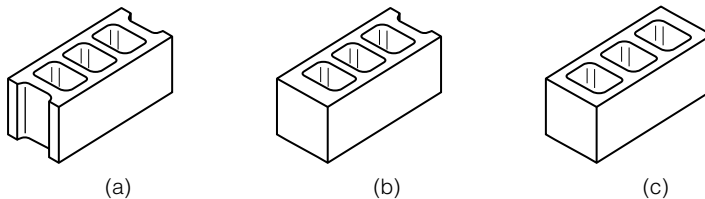


FIGURE 8.5 Concrete masonry units: (a) stretcher, (b) single-corner, and (c) double-corner.

8.1.2 ■ Clay Bricks

Clay bricks are small, rectangular blocks made of fired clay. Clays for brick making vary widely in composition from one place to another. Clays are composed mainly of silica (grains of sand), alumina, lime, iron, manganese, sulfur, and phosphates, with different proportions. Bricks are manufactured by grinding or crushing the clay in mills and mixing it with water to make it plastic. The plastic clay is then molded, textured, dried, and finally fired. Bricks are manufactured with different colors, such as dark red, purple, brown, gray, pink, or dull brown, depending on the firing temperature of the clay during manufacturing. The firing temperature for brick manufacturing varies from 900°C to 1200°C (1650°F to 2200°F). Clay bricks have an average density of 2 Mg/m³ (125 pcf).

Bricks are used for different purposes, including building, facing and aesthetics, floor making, and paving. *Building bricks (common bricks)* are used as a structural material and typically are strong and durable. *Facing bricks* are used for facing and aesthetic purposes and are available in different sizes, colors, and textures. *Floor bricks* are used on finished floor surfaces and are generally smooth and dense and have high resistance to abrasion. Finally, *paving bricks* are used as a paving material for roads, sidewalks, patios, driveways, and interior floors. Paving bricks are available in different colors, such as red, gray, or brown, and typically they are abrasion resistant and could be vitrified.

Absorption is one of the important properties that determine the durability of bricks. Highly absorptive bricks can cause efflorescence and other problems in the masonry. According to ASTM C67 absorption by 24-hour submersion, absorption by 5-hour boiling, and saturation coefficient are calculated as:

$$\text{Absorption by 24-hour submersion(\%)} = \frac{(W_{s24} - W_d)}{W_d} \times 100 \quad (8.5)$$

$$\text{Absorption by 5-hour boiling(\%)} = \frac{(W_{b5} - W_d)}{W_d} \times 100 \quad (8.6)$$

$$\text{Saturation coefficient} = \frac{(W_{s24} - W_d)}{(W_{b5} - W_d)} \quad (8.7)$$

where

- W_d = dry weight of specimen,
- W_{s24} = saturated weight after 24-hour submersion in cold water, and
- W_{b5} = saturated weight after 5-hour submersion in boiling water.

Clay bricks are very durable, fire resistant, and require very little maintenance. They have moderate insulating properties, which make brick houses cooler in summer and warmer in winter, compared with houses built with other construction materials. Clay bricks are also noncombustible and poor conductors.

The compressive strength of clay bricks is an important mechanical property that controls their load-carrying capacity and durability. The compressive strength of clay bricks is dependent on the composition of the clay, method of brick manufacturing, and the degree of firing. The compressive strength is determined by capping and testing a half unit “flatwise” (load applied in the direction of the height of the unit) and is calculated by dividing the load at failure by the cross-sectional area (ASTM C67). In determining the compressive strength, either the net or gross cross-sectional area is used. Net cross-sectional area is used only if the net cross-section is less than 75% of the gross cross section. A quarter of a brick can be tested if the capacity of the testing machine is not large enough to test a half brick. Other mechanical properties of bricks include modulus of rupture, tensile strength, and modulus of elasticity. Most clay bricks have modulus of rupture between 3.5 MPa and 26.2 MPa (500 psi and 3800 psi). The tensile strength is typically between 30% to 49% of the modulus of rupture. The modulus of elasticity ranges between 10.3 GPa and 34.5 GPa (1.5×10^6 psi and 5×10^6 psi).

Building bricks are graded according to properties related to durability and resistance to weathering, such as compressive strength, water absorption, and saturation coefficient (ASTM C62). Table 8.3 shows the three available grades and their requirements: SW, MW, and NW, standing for severe weathering, moderate weathering, and negligible weathering, respectively. Grade SW bricks are intended for use in areas subjected to frost action, especially at or below ground level. Grade NW bricks are recommended for use in areas with no frost action and in dry locations, even where subfreezing temperatures are expected. Grade NW bricks can be used in interior construction, where no freezing occurs. Note that higher compressive strengths, lower water absorptions, and lower saturation coefficients are required as the weathering condition gets more severe, so as to reduce the effect of freezing and thawing and wetting and drying.

TABLE 8.3 Physical Requirements for Building Bricks (ASTM C62) (Copyright ASTM, reprinted with permission).

Grade	Min. Compressive Strength, Gross Area, MPa (psi)		Max. Water Absorption by 5-hour Boiling, %		Max. Saturation Coefficient	
	Average of Five Bricks	Individual	Average of Five Bricks	Individual	Average of Five Bricks	Individual
SW ¹	20.7 (3000)	17.2 (2500)	17.0	20.0	0.78	0.80
MW ²	17.2 (2500)	15.2 (2200)	22.0	25.0	0.88	0.90
NW ³	10.3 (1500)	8.6 (1250)	No limit	No limit	No limit	No limit

¹ Severe weathering

² Moderate weathering

³ Negligible weathering

Sample Problem 8.3

The 5-hour boiling test was performed on a medium weathering clay brick according to ASTM C67 and produced the following masses:

dry mass of specimen = 1.788 kg

saturated mass after 5-hour submersion in boiling water = 2.262 kg

Calculate percent absorption by 5-hour boiling and check whether the brick satisfies the ASTM requirements.

Solution

$$\text{Absorption by 5-hour boiling} = \frac{2.262 - 1.788}{1.788} \times 100 = 26.5\%$$

From Table 8.3, the maximum allowable absorption by 5-hour boiling = 25.0%. Therefore, the brick does not satisfy the ASTM requirements.

Facing bricks (ASTM C216) are manufactured in two durability grades for severe weathering (SW) and moderate weathering (MW). Each durability grade is manufactured in three appearance types: FBS, FBX, and FBA. These three types stand for *face brick standard*, *face brick extra*, and *face brick architecture*. Type FBS bricks are used for general exposed masonry construction. Type FBX bricks are used for general exterior and interior masonry construction, where a high degree of precision and a low permissible variation in size are required. The FBA type bricks are manufactured to produce characteristic architectural effects resulting from nonuniformity in size and texture of the individual units.

Similar to concrete masonry units, bricks are designated by their nominal dimensions. The *nominal dimension* of the brick is greater than its *specified* (or *modular*) dimension by the thickness of the mortar joint, which is about 10 mm (3/8 in.) and could go up to 12.5 mm (1/2 in.). The *actual size* of the brick depends on the nominal size and the amount of shrinking that occurs during the firing process, which ranges from 4% to 15%.

Clay bricks are specified by their nominal width times nominal height times nominal length. For example, a $4 \times 2\text{-}2/3 \times 8$ brick has nominal width of 100 mm (4 in.), height of 70 mm (2-2/3 in.), and length of 200 mm (8 in.). Clay bricks are available in nominal widths ranging from 75 mm to 300 mm (3 in. to 12 in.), heights from 50 mm to 200 mm (2 in. to 8 in.), and lengths up to 400 mm (16 in.). Bricks can be classified as either modular or non-modular, where modular bricks have widths and lengths of multiples of 100 mm (4 in.).

8.2 Mortar

Mortar is a mixture of portland cement, lime, sand, and water. Adding a small percentage of lime to the cement mortar makes the mortar “fat” or “rich,” which increases its workability. Mortar can be classified as lime mortar or cement mortar. *Lime mortar* is made of lime, sand, and water, whereas *cement* (or *cement–lime*) *mortar* is made of lime mortar mixed with portland cement (Portland Cement Association 1987).

Mortar is used for the following functions:

- bonding masonry units together
- serving as a seating material for the units
- leveling and seating the units
- providing aesthetic quality of the structure

Lime mortar gains strength slowly with a typical compressive strength of 0.7 MPa to 2.8 MPa (100 psi to 400 psi). Cement mortar is manufactured in four types: M, S, N, and O. Type M has the lowest amount of hydrated lime, whereas type O has the highest amount. The compressive strength of mortar is tested using 50-mm cubes according to ASTM C109. The minimum average compressive strengths of types M, S, N, and O at 28 days are 17.2 MPa, 12.4 MPa, 5.2 MPa, and 2.4 MPa (2500 psi, 1800 psi, 750 psi, and 350 psi) (ASTM C270).

Mortar starts to bind masonry units when it sets. During construction, bricks and blocks should be rubbed and pressed down in order to force the mortar into the pores of the masonry units to produce maximum adhesion. It should be noted, however, that mortar is the weakest part of the masonry wall. Therefore, thin mortar layers generally produce stronger walls than do thick layers.

Unlike concrete, the compressive strength is not the most important property of mortar. Since mortar is used as an adhesive and sealant, it is very important that it forms a complete, strong, and durable bond with the masonry units and with the rebars that might be used to reinforce masonry walls. The ability to bond individual units is measured by the *tensile bond strength* of mortar (ASTM C952), which is related to the force required to separate the units. The tensile bond strength affects the shear and flexural strength of masonry. The tensile bond strength is usually between 0.14 MPa and 0.55 MPa (20 psi to 80 psi) and is affected by the amount of lime in the mix.

Other properties that affect the performance of mortar are workability, tensile strength, compressive strength, resistance to freeze and thaw, and water retentivity. The water retentivity is a measure, according to ASTM C91, of the rate at which water is lost to the masonry units.

8.3 Grout

Grout is a high-slump concrete consisting of portland cement, sand, fine gravel, water, and sometimes lime. Grout is used to fill the cores or voids in

hollow masonry units for the purpose of (1) bonding the masonry units, (2) bonding the reinforcing steel to the masonry, (3) increasing the bearing area, (4) increasing fire resistance, and (5) improving the overturning resistance by increasing the weight. The compressive strength of grout is usually about 14 MPa (2000 psi) at 28 days.

8.4 Plaster

Plaster is a fluid mixture of portland cement, lime, sand, and water, which is used for finishing either masonry walls or framed (wood) walls. Plaster is used for either exterior or interior walls. Stucco is plaster used to cover exterior walls. The average compressive strength of plaster is about 13.8 MPa (2000 psi) at 28 days.

S U M M A R Y

Masonry is one of the oldest building technologies, dating back to use of sun-dried adobe blocks in ancient times. Modern masonry units are produced to high standards in the manufacturing process. While the strength of the masonry units is important for quality control, the strength of masonry construction is generally limited by the ability to bond the units together with mortar. The ability of masonry units to resist environmental degradation is an important quality consideration. This ability is closely related to the absorption of the masonry units.

Q U E S T I O N S A N D P R O B L E M S

- 8.1 Define solid and hollow masonry units according to ASTM C90.
- 8.2 What are the advantages of masonry walls over framed (wood) walls?

- 8.3 A concrete masonry unit is tested for compressive strength and produces the following results:

$$\begin{aligned} \text{Failure load} &= 593 \text{ kN} \\ \text{Gross area} &= 0.074 \text{ m}^2 \\ \text{Gross volume} &= 0.014 \text{ m}^3 \\ \text{Net volume} &= 0.006 \text{ m}^3 \end{aligned}$$

Is the unit categorized as solid or hollow? Why? What is the compressive strength? Does the compressive strength satisfy the ASTM requirements for load bearing units shown in Table 8.2?

- 8.4 A half-block concrete masonry unit is tested for compressive strength. The outside dimensions of the specimen are 7.5 in. \times 7.5 in. \times 7.5 in. The cross section is a hollow square with a wall thickness of 1 inch. The load is applied perpendicular to the hollow cross section and the maximum load is 46,216 lb.
- Determine the gross area compressive strength.
 - Determine the net area compressive strength.
- 8.5 A concrete masonry unit has actual gross dimensions of 7-5/8" \times 7-5/8" \times 7-5/8". The unit is tested in a compression machine with the following results:

$$\begin{aligned} \text{Failure Load} &= 110 \text{ kips} \\ \text{Net volume of} &= 366.2 \text{ in}^3 \end{aligned}$$

- Is the unit categorized as solid or hollow?
 - Calculate the gross area compressive strength.
 - Calculate the net area compressive strength.
- 8.6 A half-block concrete masonry unit is subjected to compression until failure. The outside dimensions of the specimen are 190 mm \times 190 mm \times 190 mm. The cross section is a hollow square with a wall thickness of 38 mm. The load is applied perpendicular to the hollow cross section and the maximum load is 217 kN.
- Determine the gross area compressive strength.
 - Determine the net area compressive strength.
- 8.7 A symmetrical hollow concrete masonry unit, with a cross-sectional area as shown in Figure P8.7, is tested for compressive strength. The compressive load at failure is 412 KN. What is the compressive strength in MPa?

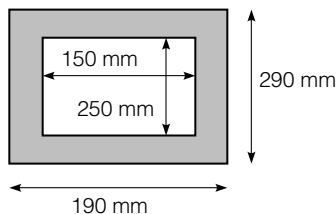


FIGURE P8.7

- 8.8 A concrete masonry unit has actual gross dimensions of $7\text{-}5/8'' \times 7\text{-}5/8'' \times 7\text{-}5/8''$. The unit is tested in a compression machine with the following results:

$$\begin{aligned} \text{Max. Failure Load} &= 83 \text{ kips} \\ \text{Net volume} &= 312.7 \text{ in}^3 \end{aligned}$$

- a. Is the unit categorized as solid or hollow? Why?
 - b. Calculate the gross area compressive strength.
 - c. Calculate the net area compressive strength.
- 8.9
- a. Why it is important that, the concrete masonry units meet certain absorption requirements?
 - b. A portion of a normal-weight concrete masonry unit was tested for absorption and moisture content and produced the following weights:

$$\begin{aligned} \text{weight of unit as received} &= 3.605 \text{ lb} \\ \text{saturated weight of unit} &= 3.939 \text{ lb} \\ \text{oven-dry weight of unit} &= 3.524 \text{ lb} \\ \text{immersed weight of unit} &= 1.684 \text{ lb} \end{aligned}$$

Calculate the absorption in lb/ft^3 and the moisture content of the unit as a percent of total absorption. Does the absorption meet the ASTM C90 requirement for absorption?

- 8.10 A portion of a medium-weight concrete masonry unit was tested for absorption and moisture content and produced the following weights:

$$\begin{aligned} \text{weight of unit as received} &= 5435 \text{ g} \\ \text{saturated weight of unit} &= 5776 \text{ g} \\ \text{oven-dry weight of unit} &= 5091 \text{ g} \\ \text{immersed weight of unit} &= 2973 \text{ g} \end{aligned}$$

Calculate the absorption in kg/m^3 and the moisture content of the unit as a percent of total absorption. Does the absorption meet the ASTM C90 requirement for absorption?

- 8.11 A portion of a concrete masonry unit was tested for absorption and moisture content according to ASTM C140 procedure and produced the following weights:

$$\begin{aligned} \text{weight of unit as received} &= 7805 \text{ g} \\ \text{saturated weight of unit} &= 8223 \text{ g} \\ \text{oven-dry weight of unit} &= 7684 \text{ g} \\ \text{immersed weight of unit} &= 4027 \text{ g} \end{aligned}$$

Determine the following:

- a. percent absorption
- b. moisture content of the unit as a percent of total absorption
- c. density
- d. weight classification according to ASTM C90 (lightweight, medium weight, or normal weight).

8.12 A concrete masonry unit was tested according to ASTM C140 procedure and produced the following results:

weight of unit as received = 8271 g
 saturated weight of unit = 8652 g
 oven-dry weight of unit = 7781 g

For this unit, calculate (a) absorption in percent and (b) moisture content as a percent of absorption

8.13 Define the nominal, specified (modular), and actual dimensions of clay bricks.

8.14 State and define the three grades of clay bricks.

8.15 A severe weathering clay brick was tested for absorption and saturation coefficient according to ASTM C67 procedure and produced the following data:

dry weight of specimen = 1.822 kg
 saturated weight after 24-hour submersion in cold
 water = 2.044 kg
 saturated weight after 5-hour submersion in boiling
 water = 2.060 kg.

Calculate absorption by 24-hour submersion, absorption by 5-hour boiling, and saturation coefficient. Does the brick satisfy the ASTM requirements?

8.16 What are the functions of mortar?

8.5 References

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9



ASPHALT BINDERS AND ASPHALT MIXTURES

Asphalt is one of the oldest materials used in construction. Asphalt binders were used in 3000 B.C., preceding the use of the wheel by 1000 years. Before the mid-1850s, asphalt came from natural pools found in various locations throughout the world, such as the Trinidad Lake asphalt, which is still mined. However, with the discovery and refining of petroleum in Pennsylvania, use of asphalt cement became widespread. By 1907, more asphalt cement came from refineries than came from natural deposits. Today, practically all asphalt cement is from refined petroleum.

Bituminous materials are classified as asphalts and tars, as shown in Figure 9.1. Several asphalt products are used; asphalt is used mostly in pavement construction, but is also used as sealing and waterproofing agents. Tars are produced by the destructive distillation of bituminous coal or by cracking petroleum vapors. In the United States, tar is used primarily for waterproofing membranes, such as roofs. Tar may also be used for pavement treatments, particularly where fuel spills may dissolve asphalt cement, such as on parking lots and airport aprons.

The fractional distillation process of crude petroleum is illustrated in Figure 9.2. Different products are separated at different temperatures. Figure 9.2 shows the main products, such as gasoline, kerosene, diesel oil, and asphalt residue (asphalt cement). Since asphalt is a lower-valued product than other components of crude oil, refineries are set up to produce the more valuable fuels at the expense of asphalt production. The quantity and quality of the asphalt depends on the crude petroleum source and the refining method. Some crude sources, such as the Nigerian oils, produce little asphalt, while others, such as many of the Middle Eastern oils, have a high asphalt content.

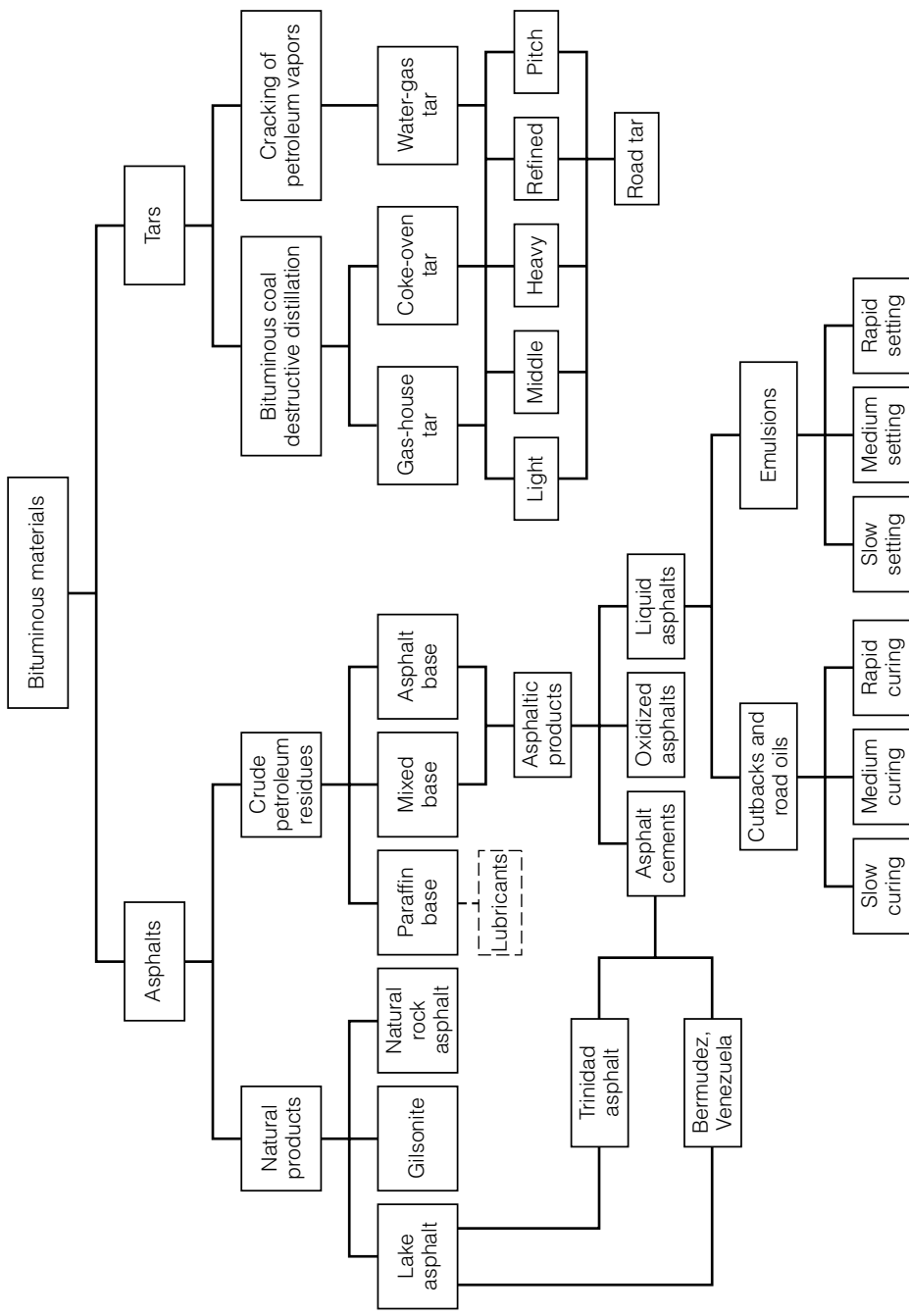


FIGURE 9.1 Classification of bituminous materials. (Goetz and Wood 1960).

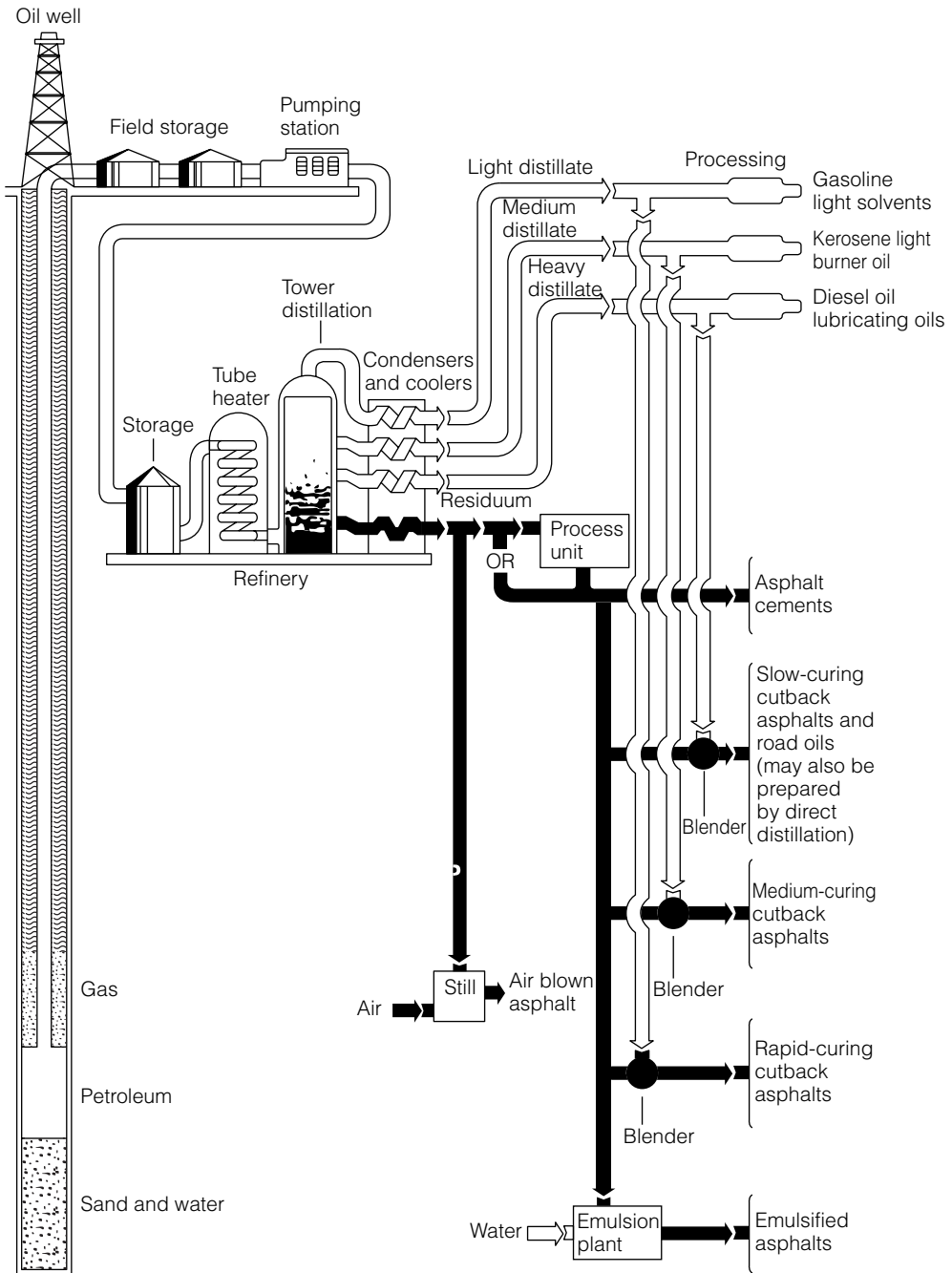


FIGURE 9.2 Distillation of crude petroleum. (The Asphalt Institute 1989).

This chapter reviews the types, uses, and chemical and physical properties of asphalt. The asphalt concrete used in road and airport pavements, which is a mixture of asphalt and aggregates, is also presented. The chapter discusses the recently developed Performance Grade asphalt binder specifications and Superpave mix design. Recycling of pavement materials and additives used to modify the asphalt properties are also included.

9.1 Types of Asphalt Products

Asphalt used in pavements is produced in three forms: *asphalt cement*, *asphalt cutback*, and *asphalt emulsion*. Asphalt cement is a blend of hydrocarbons of different molecular weights. The characteristics of the asphalt depend on the chemical composition and the distribution of the molecular weight hydrocarbons. As the distribution shifts toward heavier molecular weights, the asphalt becomes harder and more viscous. At room temperatures, asphalt cement is a semisolid material that cannot be applied readily as a binder without being heated. Liquid asphalt products, cutbacks and emulsions, have been developed and can be used without heating (The Asphalt Institute 1989).

Although the liquid asphalts are convenient, they cannot produce a quality of asphalt concrete comparable to what can be produced by heating neat asphalt cement and mixing it with carefully selected aggregates. Asphalt cement has excellent adhesive characteristics, which make it a superior binder for pavement applications. In fact, it is the most common binder material used in pavements.

A cutback is produced by dissolving asphalt cement in a lighter molecular weight hydrocarbon solvent. When the cutback is sprayed on a pavement or mixed with aggregates, the solvent evaporates, leaving the asphalt residue as the binder. In the past, cutbacks were widely used for highway construction. They were effective and could be applied easily in the field. However, three disadvantages have severely limited the use of cutbacks. First, as petroleum costs have escalated, the use of these expensive solvents as a carrying agent for the asphalt cement is no longer cost effective. Second, cutbacks are hazardous materials due to the volatility of the solvents. Finally, application of the cutback releases environmentally unacceptable hydrocarbons into the atmosphere. In fact, many regions with air pollution problems have outlawed the use of any cutback material.

An alternative to dissolving the asphalt in a solvent is dispersing the asphalt in water as emulsion. In this process the asphalt cement is physically broken down into micron-sized globules that are mixed into water containing an emulsifying agent. Emulsified asphalts typically consist of about 60% to 70% asphalt residue, 30% to 40% water, and a fraction of a percent of

emulsifying agent. There are many types of emulsifying agents; basically they are a soap material. The emulsifying molecule has two distinct components, the head portion, which has an electrostatic charge, and the tail portion, which has a high affinity for asphalt. The charge can be either positive to produce a *cationic* emulsion or negative to produce an *anionic* emulsion. When asphalt is introduced into the water with the emulsifying agent, the tail portion of the emulsifier attaches itself to the asphalt, leaving the head exposed. The electric charge of the emulsifier causes a repulsive force between the asphalt globules, which maintains their separation in the water. Since the specific gravity of asphalt is very near that of water, the globules have a neutral buoyancy and, therefore, do not tend to float or sink. When the emulsion is mixed with aggregates or used on a pavement, the water evaporates, allowing the asphalt globs to come together, forming the binder. The phenomenon of separation between the asphalt residue and water is referred to as *breaking* or *setting*. The rate of emulsion setting can be controlled by varying the type and amount of the emulsifying agent.

Since most aggregates bear either positive surface charges (such as limestone) or negative surface charges (such as siliceous aggregates), they tend to be compatible with anionic or cationic emulsions, respectively. However, some emulsion manufacturers can produce emulsions that bond well to aggregate-specific types, regardless of the surface charges.

Although emulsions and cutbacks can be used for the same applications, the use of emulsions is increasing because they do not include hazardous and costly solvents.

9.2 Uses of Asphalt

The main use of asphalt is in pavement construction and maintenance. In addition, asphalt is used in sealing and waterproofing various structural components, such as roofs and underground foundations.

The selection of the type and grade of asphalt depends on the type of construction and the climate of the area. Asphalt cements, also called asphalt binders, are used typically to make hot-mix asphalt concrete for the surface layer of asphalt pavements (See Figures 9.3 and 9.4). Asphalt concrete is also used in patching and repairing both asphalt and portland cement concrete pavements. Liquid asphalts (emulsions and cutbacks) are used for pavement maintenance applications, such as fog seals, chip seals, slurry seals, and microsurfacing (See Figures 9.5 and 9.6) (The Asphalt Institute 1989, Mamlouk and Zaniewski 1998). Liquid asphalts may also be used to seal the cracks in pavements. Liquid asphalts are mixed with aggregates to produce cold mixes, as well. Cold mixtures are normally used for patching (when hot-mix asphalt concrete is not available), base and subbase stabilization, and surfacing of low-volume roads. Table 9.1 shows common paving applications for asphalts.



FIGURE 9.3 Placing hot mix asphalt (asphalt concrete) used for the surface layer of asphalt pavement.



FIGURE 9.4 Compaction of hot mix asphalt.



FIGURE 9.5 Applying fog seal for preserving existing pavement.



FIGURE 9.6 Applying microsurfacing for preserving existing pavement.

TABLE 9.1 Paving Applications of Asphalt

Term	Description	Application
Hot mix asphalt	Carefully designed mixture of asphalt and aggregates	Pavement surface, patching
Cold mix	Mixture of aggregates and liquid asphalt	Patching, low volume road surface, asphalt stabilized base
Fog seal	Spray of diluted asphalt emulsion on existing pavement surface	Seal existing pavement surface
Prime coat	Spray coat to bond aggregate base and asphalt concrete surface	Construction of flexible pavement
Tack coat	Spray coat between lifts of asphalt concrete	Construction of new pavements or between an existing pavement and an overlay
Chip seal	Spray coat of asphalt emulsion (or asphalt cement or cutback) followed with aggregate layer	Maintenance of existing pavement or low volume road surfaces
Slurry seal	Mixture of emulsion, well-graded fine aggregate and water	Resurface low volume roads
Microsurfacing	Mixture of polymer modified emulsion, well-graded crushed fine aggregate, mineral filler, water, and additives	Texturing, sealing, crack filling, rut filling, and minor leveling

9.3 Temperature Susceptibility of Asphalt

The consistency of asphalt is greatly affected by temperature. Asphalt gets hard and brittle at low temperatures and soft at high temperatures. Figure 9.7 shows a conceptual relation between temperature and logarithm of viscosity. The viscosity of the asphalt decreases when the temperature increases. Asphalt's temperature susceptibility can be represented by the slope of the line shown in Figure 9.7. The steeper the slope the higher the temperature susceptibility of the asphalt. However, additives can be used to reduce this susceptibility.

When asphalt is mixed with aggregates, the mixture will perform properly only if the asphalt viscosity is within an optimum range. If the viscosity of asphalt is higher than the optimum range, the mixture will be too

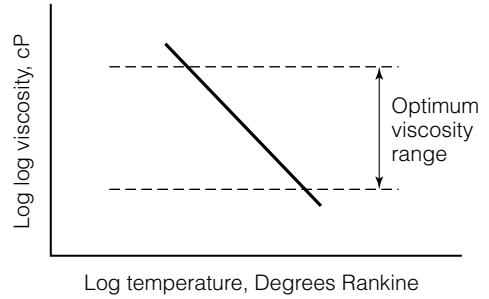


FIGURE 9.7 Typical relation between asphalt viscosity and temperature.



FIGURE 9.8 Thermal cracking resulting from the use of too stiff asphalt in a cold climate area.

brittle and susceptible to low-temperature cracking (Figure 9.8). On the other hand, if the viscosity is below the optimum range, the mixture will flow readily, resulting in permanent deformation (rutting) as shown in Figure 9.9.

Due to temperature susceptibility, the grade of the asphalt cement should be selected according to the climate of the area. The viscosity of the asphalt should be mostly within the optimum range for the area's annual temperature range; soft-grade asphalts are used for cold climates and hard-grade asphalts for hot climates (See Figure 9.10).



FIGURE 9.9 Rutting that could result from the use of too soft asphalt.

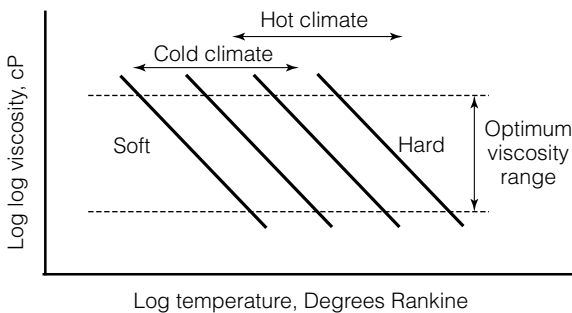


FIGURE 9.10 Selecting the proper grade of asphalt binder to match the climate.

9.4 Chemical Properties of Asphalt

Asphalt is a mixture of a wide variety of hydrocarbons primarily consisting of hydrogen and carbon atoms, with minor components such as sulfur, nitrogen, and oxygen (heteroatoms), and trace metals. The percentages of the chemical components, as well as the molecular structure of asphalt, vary depending on the crude oil source (Peterson 1984).

The molecular structure of asphalt affects the physical and aging properties of asphalt, as well as how the asphalt molecules interact with each other and with aggregate. Asphalt molecules have three arrangements, depending on the carbon atom links: (1) *aliphatic* or paraffinic, which form straight or branched chains, (2) *saturated rings*, which have the highest hydrogen to carbon ratio, and (3) *unsaturated rings* or aromatic. Heteroatoms attached to carbon alter the molecular configuration. Since the number of molecular structures of asphalt is extremely large, research on asphalt chemistry has focused on separating asphalt into major fractions that are less complex or more homogeneous. Each of these fractions is a complex chemical structure.

Asphalt cement consists of asphaltenes and maltenes (petrolenes). The maltenes consist of resins and oils. The asphaltenes are dark brown friable solids that are chemically complex, with the highest polarity among the components. The asphaltenes are responsible for the viscosity and the adhesive property of the asphalt. If the asphaltene content is less than 10%, the asphalt concrete will be difficult to compact to the proper construction density. Resins are dark and semisolid or solid, with a viscosity that is largely affected by temperature. The resins act as agents to disperse asphaltenes in the oils; the oils are clear or white liquids. When the resins are oxidized, they yield asphaltene-type molecules. Various components of asphalt interact with each other to form a balanced or compatible system. This balance of components makes the asphalt suitable as a binder.

Three fractionation schemes are used to separate asphalt components, as illustrated in Figure 9.11. The first scheme [Figure 9.11(a)] is partitioning with partial solvents in which *n*-butanol is added to separate (precipitate) the asphaltics. The butanol is then evaporated, and the remaining component is dissolved in acetone and chilled to -23°C to precipitate the paraffinics and leave the cyclics in solution. The second scheme [Figure 9.11(b)] is selective adsorption–desorption, in which *n*-heptane is added to separate asphaltene. The remaining maltene fraction is introduced to a chromatographic column and desorbed using solvents with increasing polarity to separate other fractions. The third scheme [Figure 9.11(c)] is chemical precipitation in which *n*-pentane is added to separate the asphaltenes. A sulfuric acid (H_2SO_4) is added in increasing strengths to precipitate other fractions.

In addition, asphalt can be separated based on the molecular size with the use of high-pressure liquid chromatography (gel-permeation chromatography).

9.5 Superpave and Performance Grade Binders

In 1987, the Strategic Highway Research Program (SHRP) began developing a new system for specifying asphalt materials and designing asphalt mixes.

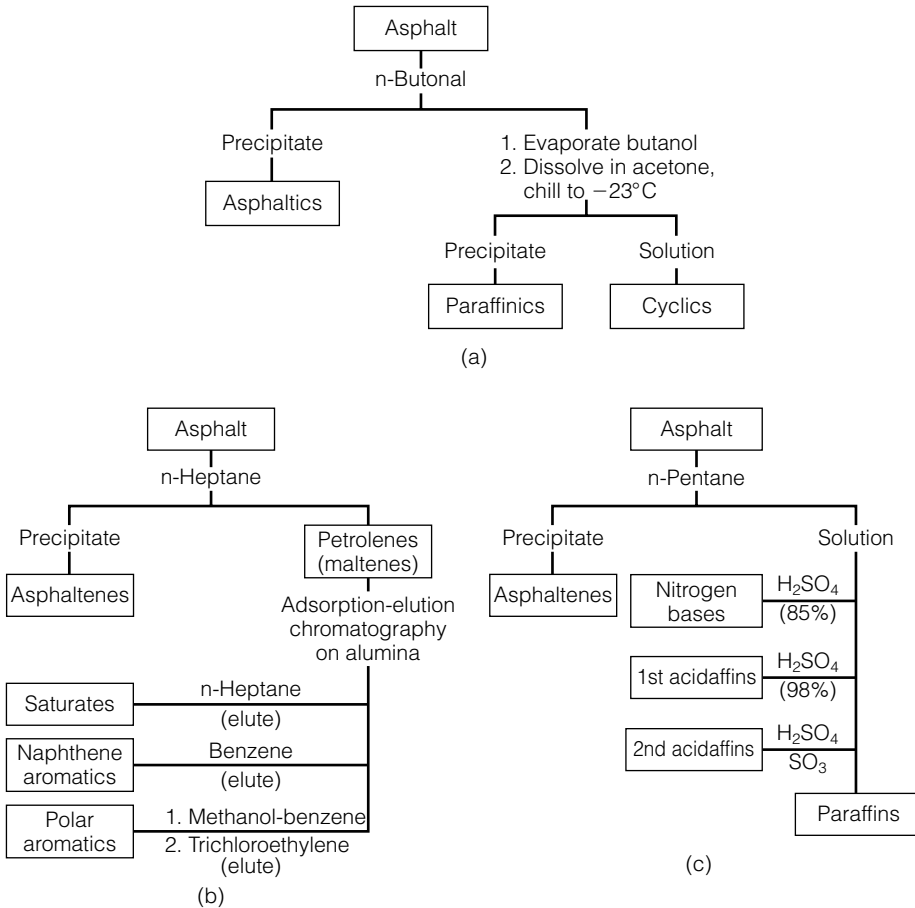


FIGURE 9.11 Schematic diagrams of three asphalt fractionation schemes: (a) partitioning with partial solvents, (b) selective-adsorption-description, and (c) chemical precipitation (Peterson 1984).

The SHRP research program produced the Superpave (*Superior Performing Asphalt Pavements*) mix design method for asphalt concrete and the Performance Grading method for asphalt binder specification (McGennis 1994; 1995). The objectives of SHRP’s asphalt research were to extend the life or reduce the life-cycle costs of asphalt pavements, to reduce maintenance costs, and to minimize premature failures. An important result of this research effort was the development of performance-based specifications for asphalt binders and mixtures to control three distress modes: rutting, fatigue cracking, and thermal cracking. Note that the Performance Grade specifications use the term *asphalt binder*, which refers to asphalt cement with or without the addition of modifiers.

9.6 Characterization of Asphalt

Many tests are available to characterize asphalt cement. Some tests are commonly used by highway agencies, while others are used for research. Since the properties of the asphalt are highly sensitive to temperature, all asphalt tests must be conducted at a specified temperature within very tight tolerances (The Asphalt Institute 1989).

Before the SHRP research, the asphalt cement specifications typically were based on measurements of viscosity, penetration, ductility, and softening point temperature. These measurements are not sufficient to properly describe the viscoelastic and failure properties of asphalt cement that are needed to relate asphalt binder properties to mixture properties and to pavement performance. The new Performance Grade binder specifications were designed to provide performance-related properties that can be related in a rational manner to pavement performance (McGennis 1994).

9.6.1 Performance Grade Characterization Approach

The Performance Grade tests used to characterize the asphalt binder are performed at pavement temperatures to represent the upper, middle, and lower range of service temperatures. The measurements are obtained at temperatures in keeping with the distress mechanisms. Therefore, unlike previous specifications that require performing the test at a fixed temperature and varying the requirements for different grades of asphalt, the Performance Grade specifications require performing the test at the critical pavement temperature and fixing the criteria for all asphalt grades. Thus, the Performance Grade philosophy ensures that the asphalt properties meet the specification criteria at the critical pavement temperature.

Three pavement design temperatures are required for the binder specifications: a maximum, an intermediate, and a minimum temperature. The maximum and minimum pavement temperatures for a given geographical location in the United States can be generated using algorithms contained within the SHRP software, based on weather information from 7500 weather stations. The maximum pavement design temperature is selected as the highest successive seven-day average maximum pavement temperature. The minimum pavement design temperature is the minimum pavement temperature expected over the life of the pavement. The intermediate pavement design temperature is the average of the maximum and minimum pavement design temperatures plus 4°C.

Laboratory tests that evaluate rutting potential use the maximum pavement design temperature, whereas tests that evaluate fatigue potential use the intermediate pavement design temperature. Thermal-cracking tests use the minimum pavement design temperature plus 10°C (18°F). The minimum pavement design temperature is increased by 10°C to reduce the testing

time. These results are corrected to the minimum temperature using the time-temperature shift factor (McGennis 1994).

9.6.2 ■ Performance Grade Binder Characterization

Several tests are used in the Performance Grade method to characterize the asphalt binder. Some of these tests have been used before for asphalt testing, while others are new. The discussion that follows summarizes the main steps and the significance of SHRP tests. With the exception of the rotational (Brookfield) viscometer, solubility, and flash point tests, the test temperatures are selected based on the temperature at the design location. The binder specification indicates the specific test temperatures used for various binders for each test (McGennis 1994). Four tests are performed on the neat or tank asphalt: flash point, solubility, rotational viscosity, and dynamic shear rheometer. To simulate the effect of aging on the properties of the binder, the rolling thin-film oven and pressure-aging vessel condition the binder for short-term and long-term effects. Samples are conditioned with both the rolling thin film oven and pressure-aging vessel before determining their characteristics with respect to fatigue and low-temperature cracking.

Rolling Thin-Film Oven The rolling thin-film oven (RTFO) procedure is used to simulate the short-term aging that occurs in the asphalt during production of asphalt concrete. In the RTFO method (ASTM D2872), the asphalt binder is poured into special bottles, as shown in Figure 9.12. The bottles are placed in a rack in a forced-draft oven at a temperature of 163°C (325°F) for 75 min. The rack rotates vertically, continuously exposing fresh asphalt. The binder in the rotating bottles is also subjected to an air jet to speed up the aging process. The Performance Grade specifications limit the amount of mass loss during RTFO conditioning. Rolling thin-film oven conditioning is used to prepare samples for evaluation for rutting potential with the dynamic shear



FIGURE 9.12 Rolling thin film oven test apparatus.

rheometer and prior to conditioning with the pressure-aging vessel. Under the penetration and viscosity grading methods, the aged binder is usually tested for penetration or viscosity and the results are compared with those of new asphalt.

Pressure-Aging Vessel The pressure-aging vessel (PAV) consists of a temperature-controlled chamber, and pressure- and temperature-controlling and measuring devices, as illustrated in Figure 9.13 (ASTM D6521). The asphalt binder is first aged, using the rolling thin-film oven (RTFO) (ASTM D2872). A specified thickness of residue from the RTFO is placed in the PAV pans. The asphalt is then aged at the specified aging temperature for 20 hours in a vessel under 2.10 MPa (305 psi) of air pressure. Aging temperature, which ranges between 90°C and 110°C, is selected according to the grade of the asphalt binder. Since the procedure forces oxygen into the sample, it is necessary to use a vacuum oven to remove any air bubbles from the sample prior to testing.

The PAV is designed to simulate the oxidative aging that occurs in asphalt binders during pavement service. Residue from this process may be used to estimate the physical or chemical properties of an asphalt binder after 5 to 10 years in the field.

Flash Point At high temperatures, asphalt can flash or ignite in the presence of open flame or spark. The flash point test is a safety test that measures the temperature at which the asphalt flashes; asphalt cement may be heated to



FIGURE 9.13 Pressure aging vessel apparatus.

a temperature below this without becoming a fire hazard. The Cleveland open cup method (ASTM D92) requires partially filling a standard brass cup with asphalt cement. The asphalt is then heated at a specified rate and a small flame is periodically passed over the surface of the cup, as shown in Figure 9.14. The flash point is the temperature of the asphalt when the volatile fumes coming off the sample will sustain a flame for a short period of time. The minimum temperature at which there are sufficient volatile fumes to sustain a flame for an extended period of time is the fire point.

Rotational Viscometer Test The rotational (Brookfield) viscometer test (ASTM D4402) consists of a rotational coaxial cylinder viscometer and a temperature control unit, as shown in Figure 9.15. The test is performed on unaged binders. The asphalt binder sample is placed in the sample chamber at 135°C (275°F); then both are placed in the thermocell. A spindle is placed in the asphalt sample and rotated at a specified speed. The viscosity is determined by the amount of torque required to rotate the spindle at the specified speed. The spindle size used is determined based on the viscosity being measured. The viscosity is computed in centipoises (cP) by the testing machine. The Performance Grade specification limit is stated in Pascal seconds ($\text{Pa} \cdot \text{s}$), which is equal to cP divided by 1000. The viscosity is recorded as the average of three readings, at one-minute intervals, to the nearest 0.1 $\text{Pa} \cdot \text{s}$. In addition to testing at the temperature required by the specification, additional tests are performed at higher temperatures to establish the temperature susceptibility relationship used to determine the compaction and mixing temperatures required for the mix design process.

Dynamic Shear Rheometer Test The dynamic shear rheometer test system, Figure 9.16, consists of two parallel metal plates, an environmental chamber, a loading device, and a control and data acquisition system (AASHTO T315). The dynamic shear rheometer is used to measure three specification requirements in the Performance Grading system. For testing the neat binder

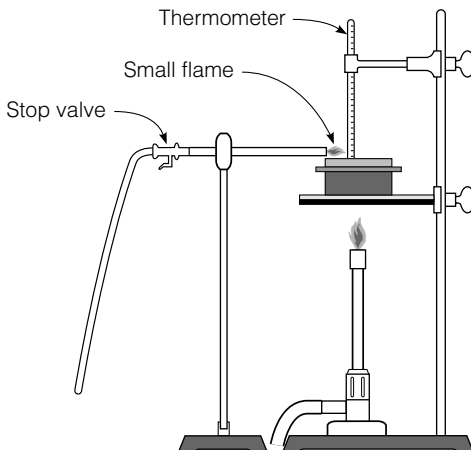


FIGURE 9.14 Cleveland open cup flash point test apparatus.



FIGURE 9.15 Rotational viscometer.



FIGURE 9.16 Dynamic shear rheometer apparatus.

and for the rutting potential test, the test temperature is equal to the upper temperature for the grade of the asphalt binder (e.g., a PG 64–22 is tested at 64°C). For these tests, the sample size is 25 mm in diameter and 1 mm thick. Prior to testing for rutting potential, the sample is conditioned in the rolling thin-film oven. For evaluating fatigue potential, the intermediate temperature is used; 25°C for PG 64–22. The sample size is 8 mm in diameter by 2 mm thick. Prior to testing, the sample is conditioned in the rolling thin-film oven, followed by the pressure-aging vessel.

During testing, one of the parallel plates is oscillated with respect to the other at preselected frequencies and rotational deformation amplitudes (or torque amplitudes). The required amplitude depends upon the value of the complex shear modulus of the asphalt binder being tested. Specification testing is performed at an angular frequency of 10 rads/s. The complex shear modulus (G^*) and phase angle (δ) are calculated automatically by the rheometer's computer software. The complex shear modulus and the phase angle define the asphalt binder's resistance to shear deformation in the linear viscoelastic region.

Bending Beam Rheometer Test The bending beam rheometer measures the midpoint deflection of a simply supported prismatic beam of asphalt binder subjected to a constant load applied to its midpoint (ASTM D6648). The bending beam rheometer test system consists of a loading frame, a controlled temperature bath, and a computer-controlled automated data acquisition unit, as shown in Figure 9.17. The test temperature is 10°C higher than the lower temperature rating of the binder (e.g., a PG 64–22 is tested at -12°C). The sample is conditioned in both the rolling thin-film oven and pressure-aging vessel prior to testing. An asphalt binder beam is placed in the bath and loaded with a constant force of 980 ± 50 mN for 240 seconds. As the beam creeps, the midpoint deflection is monitored after 8, 15, 30, 60, 120, and 240 seconds. The constant maximum stress in the beam is calculated from the load magnitude and the dimensions of the beam. The maximum strain is calculated from the deflection and the dimensions of the beam. The flexural creep stiffness of the beam is then calculated by dividing the maximum stress by the maximum strain for each of the specified loading times.

The low-temperature thermal-cracking performance of paving mixtures is related to the creep stiffness, defined as the slope of the logarithm of the creep stiffness versus the logarithm of the time curve of the asphalt binder contained in the mix.

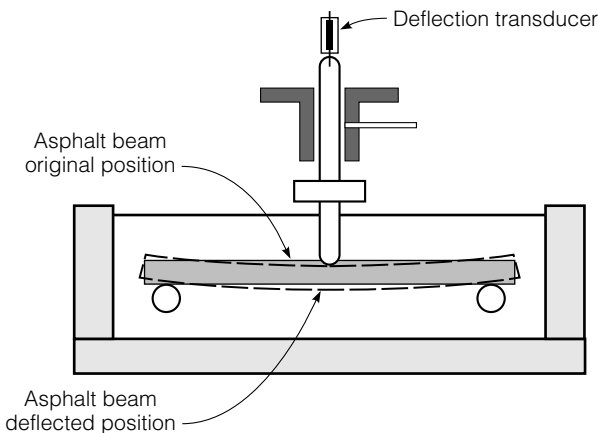


FIGURE 9.17 Schematic of the bending beam rheometer.

Direct Tension Test The direct tension test system consists of a displacement-controlled tensile loading machine with gripping system, a temperature controlled chamber, measuring devices, and a data acquisition system, as shown in Figure 9.18 (ASTM D6723). In this test, an asphalt binder specimen, conditioned in the rolling thin-film oven and pressure-aging vessel, is pulled at a constant rate of deformation of 1 mm/min. The test temperature equals the low temperature rating plus 10°C. A noncontact extensometer measures the elongation of the specimen. The maximum load developed during the test is monitored. The tensile strain and stress in the specimen when the load reaches a maximum is reported as the failure strain and failure stress, respectively.

The strain at failure is a measure of the amount of elongation that the asphalt binder can sustain without cracking. Strain at failure is used as a criterion for specifying the low-temperature properties of the binder.



FIGURE 9.18 Direct tension test apparatus.

9.6.3 ■ Traditional Asphalt Characterization Tests

Traditional tests that have been used to characterize asphalt before the development of the Performance Grade system include the penetration and absolute and kinematic viscosities.

Penetration The penetration test (ASTM D5) measures asphalt cement consistency. An asphalt sample is prepared and brought to 25°C (77°F). A standard needle with a total mass of 100 g is placed on the asphalt surface. The needle is released and allowed to penetrate the asphalt for 5 seconds, as shown in Figure 9.19. The depth of penetration, in units of 0.1 mm, is recorded and reported as the penetration value. A large penetration value indicates soft asphalt.

Absolute and Kinematic Viscosity Tests Similar to the penetration test, the viscosity test is used to measure asphalt consistency. Two types of viscosity are commonly measured: absolute and kinematic. The absolute viscosity procedure (ASTM D2171) requires heating the asphalt cement and pouring it into a viscometer placed in a water or oil bath at a temperature of 60°C (140°F) (Figure 9.20). The viscometer is a U-tube, with a reservoir where the asphalt is introduced and a section with a calibrated diameter and timing marks. For absolute viscosity tests vacuum is applied at one end. The time during which the asphalt flows between two timing marks on the viscometer is measured using a stopwatch. The flow time, measured in seconds, is multiplied by the viscometer calibration factor to obtain the absolute viscosity in units of poises. Different-sized viscometers are used for different asphalt grades to meet minimum and maximum flow time requirements of the test procedure.

The kinematic viscosity test procedure (ASTM D2170) is similar to that of the absolute viscosity test, except that the test temperature is 135°C (275°F). Since the viscosity of the asphalt at 135°C is fairly low, vacuum is not used. The time it takes the asphalt to flow between the two timing marks is multiplied by the calibration factor to obtain the kinematic viscosity in units of centistokes (cSt).

9.6.4 ■ Characterization of Emulsion and Cutback

Common methods used to characterize emulsion include distillation and Saybolt–Furol viscosity tests. Cutback is characterized by distillation.

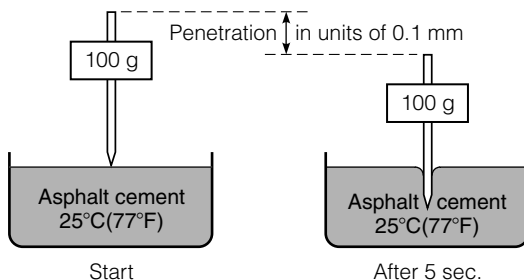


FIGURE 9.19 Penetration test.



FIGURE 9.20 Absolute viscosity test apparatus.

Distillation of Cutback and Emulsion The distillation test of cutback asphalt (ASTM D402) measures the amount and character of volatile constituents it contains. The procedure requires that the percentages, by volume, of the distillate fractions at specified temperatures be determined. The distillation test of emulsified asphalt (ASTM D244) determines the percent of residue and oil distillates by weight.

Saybolt Furol Viscosity of Emulsion Emulsion viscosity is an important factor in field applications. When applied in a spray, the emulsion must be thin enough to be uniformly applied through the spray bar of the distributor truck, yet viscous enough that it will not flow from the crown or grade of the road. Emulsion viscosity is measured using the Saybolt–Furol viscometer, as shown in Figure 9.21 (ASTM D244). In this test, the emulsion is brought to a temperature of either 25°C (77°F) or 50°C (122°F) and allowed to flow through a specific orifice. The Saybolt–Furol viscosity is the time (in seconds) required to fill a special flask.



FIGURE 9.21 Saybolt Furol viscosity.

9.7 Classification of Asphalt

Several methods are used to characterize asphalt binders, asphalt cutbacks, and asphalt emulsions.

9.7.1 Asphalt Binders

Asphalt binder is produced in several grades or classes. There are four methods for classifying asphalt binders:

1. performance grading
2. penetration grading
3. viscosity grading
4. viscosity of aged residue grading

Performance Grade Specifications and Selection Several grades of binder are available, based on their performance in the field. Names of grades start with PG (Performance Graded) followed by two numbers representing the maximum and minimum pavement design temperatures in Celsius. For example, an asphalt binder PG 52–28 would meet the specification for a design high pavement temperature up to 52°C (126°F) and a design low temperature warmer than –28°C (–18°F). These temperatures are calculated 20 mm (0.75 in.) below the pavement surface. The high and low pavement temperatures are related to the air temperature as well as other factors. Table 9.2 shows the binder grades in the Performance Grade specifications. PG 76 and 82 are intended to accommodate only slow transient or standing loads, such as those that occur near intersections or in truck climbing lanes.

The performance-graded asphalt binder specifications are shown in Table 9.3 (ASTM D6373). The table shows the design criteria of various test

TABLE 9.2 Binder Grades in the Performance Grade Specifications

High Temperature Grades (°C)	Low Temperature Grades (°C)
PG 46	–34, –40, –46
PG 52	–10, –16, –22, –28, –34, –40, –46
PG 58	–16, –22, –28, –34, –40
PG 64	–10, –16, –22, –28, –34, –40
PG 70	–10, –16, –22, –28, –34, –40
PG 76	–10, –16, –22, –28, –34
PG 82	–10, –16, –22, –28, –34

TABLE 9.3 Performance Graded Asphalt Binder Specifications

Performance Grade	PG 46-					PG 52-					PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average seven-day Maximum Pavement Design Temperature, °C	<46					<52					<58					<64					
Minimum Pavement Design Temperature, °C	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40
Flash Point Temp: Minimum °C	Original Binder																				
Flash Point Temp: Minimum °C	230																				
Viscosity, ASTM D4402: Maximum, 3Pa.s, Test Temp, °C	135																				
Dynamic Shear: G*/sin δ, Minimum, 1.00kPa Test Temp @ 10 rad/s, °C	46					52					58					64					
Mass Loss, Maximum, Percent	Rolling Thin-Film Oven Residue																				
Mass Loss, Maximum, Percent	1.00																				
Dynamic Shear: G*/Sin δ, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46					52					58					64					
PAV Aging Temperature, °C	90					90					100					100					
Dynamic Shear: G* · sin δ, Maximum, 5000 kPa Test Temp @ 10rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Creep Stiffness: S, Maximum, 300 MPa, m-value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension: Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

TABLE 9.3 (Continued)

Performance Grade	PG 70-						PG 76-						PG 82-					
	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34		
Average seven-day Maximum Pavement Design Temperature, °C	<70						<76						<82					
Minimum Pavement Design Temperature, °C	> -10	> -16	> -22	> -28	> -34	> -40	> -10	> -16	> -22	> -28	> -34	> -10	> -16	> -22	> -28	> -34		
Original Binder																		
Flash Point Temp: Minimum °C							230											
Viscosity, ASTM D4402: Maximum, 3 Pa.s, Test Temp, °C							135											
Dynamic Shear: G*/ sin δ, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	70						76						82					
Rolling Thin-Film Oven Residue																		
Mass Loss, Maximum, percent							1.00											
Dynamic Shear: G*/ sin δ, Minimum, 2.20 kPa Test Temp@ 10 rad/s, °C	70						76						82					
Pressure-aging Vessel (PAV) Residue																		
PAV Aging Temperature, °C	100 (110)						100 (110)						100 (110)					
Dynamic Shear, G* • sin δ, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28		
Creep Stiffness: S, Maximum, 300 MPa, m-value, Minimum, 0.300 Test Temp @ 60s, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24		
Direct Tension: Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24		

parameters at the specified test temperatures. One important difference between the Performance Grade specifications and the traditional specifications is in the way the specifications work. As shown in Table 9.3, the physical properties (criteria) remain constant for all grades, but the temperatures at which these properties must be achieved vary, depending on the climate at which the binder is expected to be used. The temperature ranges shown in Table 9.3 encompass all pavement temperature regimes that exist in the United States and Canada.

The binder is selected to satisfy the maximum and minimum design pavement temperature requirements. The average seven-day maximum pavement temperature is used to determine the design maximum, whereas the design minimum pavement temperature is the lowest pavement temperature. Since the maximum and minimum pavement temperatures vary from one year to another, a reliability level is considered. As used in the Performance Grade, reliability is the percent probability in a single year that the actual pavement temperature will not exceed the design high pavement temperature or be lower than the design low pavement temperature.

It is assumed that the design high and design low pavement temperatures throughout the years follow normal distributions as illustrated in Figure 9.22(a). In this example, the average seven-day maximum pavement temperature is 56°C and the standard deviation is 2°C . Similarly, the average one-day minimum pavement temperature is -23°C and the standard deviation is 4°C . Since the area under the normal distribution curve represents the probability as illustrated in Figure 1.19, the range of temperature that satisfies the assumed probability can be calculated. For example, the range between -23°C and 56°C results in a 50% reliability for both high and low temperatures. By subtracting two standard deviations from the minimum pavement temperature and adding two standard deviations to the maximum pavement temperature, the range between -31°C and 60°C results in 98% reliability. In selecting the appropriate grade, the designer should select the standard PG grade that most closely satisfies the required reliability level. This “rounding” typically results in a higher reliability level than is intended, as shown in Figure 9.22(b). Note that the reliability levels at the high- and low-temperature grades do not need to be the same, depending on the specific pavement conditions.

Sample Problem 9.1

What standard PG asphalt binder grade should be selected under the following conditions:

The seven-day maximum pavement temperature has a mean of 57°C and a standard deviation of 2°C .

The minimum pavement temperature has a mean of -6°C and a standard deviation of 3°C .

Reliability is 98%.

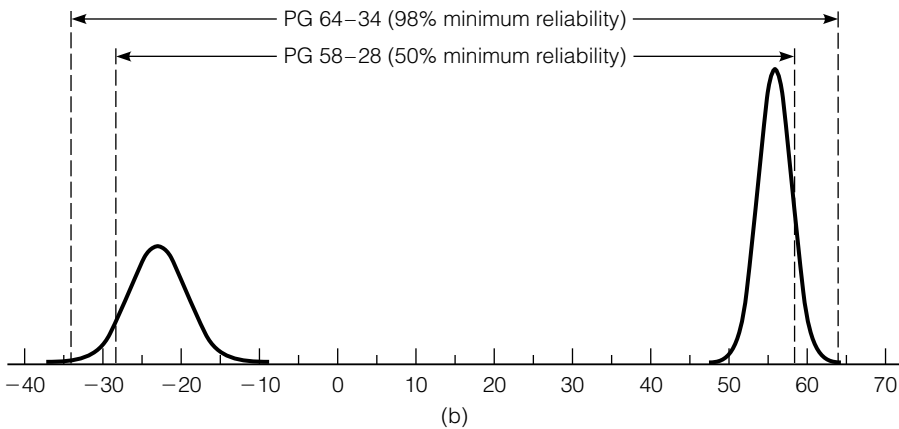
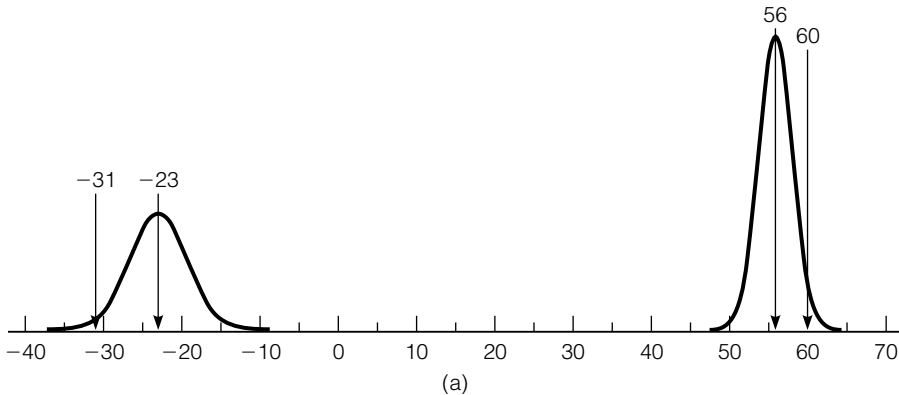


FIGURE 9.22 Example of the distribution of design pavement temperatures and selection of binder grades: (a) distribution of high and low design pavement temperatures, and (b) binder grade selection.

Solution

$$\begin{aligned} \text{High-temperature grade} &\geq 57 + (2 \times 2) \geq 61^\circ\text{C} \\ \text{Low-temperature grade} &\leq -6 - (2 \times 3) \leq -12^\circ\text{C} \end{aligned}$$

The closest standard PG asphalt binder grade that satisfies the two temperature grades is PG 64–16.

Other Asphalt Binder Grading Methods Table 9.4 shows various asphalt cement grades based on penetration and on their properties (ASTM D946). The grades correspond to the allowable penetration range; that is, the penetration of a 40–50 grade must be in the range of 40 to 50. Various grades based on viscosity and their properties are shown in Table 9.5 (ASTM D3381). The AC

TABLE 9.4 Penetration Grading System of Asphalt Cement

Grade	Penetration		Flash Point °C (°F)	Ductility (cm)
	min.	max.		
40–50	40	50	232 (450)	100
60–70	60	70	232 (450)	100
85–100	85	100	232 (450)	100
120–150	120	150	219 (425)	100
200–300	200	300	177 (350)	100

TABLE 9.5 Viscosity Grading System of Asphalt Cement

Grade	Viscosity		Penetration*	Flash Point* °C (°F)
	Absolute (poises)	Kinematic* (cSt)		
AC-2.5	250 ± 50	125	220	163 (325)
AC-5	500 ± 100	175	140	177 (350)
AC-10	1000 ± 200	250	80	219 (425)
AC-20	2000 ± 400	300	60	232 (450)
AC-30	3000 ± 600	350	50	232 (450)
AC-40	4000 ± 800	400	40	232 (450)

*Specification is for the minimum acceptable values

grade numbers are 1/100 of the middle of the allowable viscosity range; that is, an AC-5 has an absolute viscosity of 500 ± 100 poises. Therefore, high viscosity asphalt cements have a high designation number. Aged-residue grades are based on the absolute viscosity of the asphalt after it has been conditioned to simulate the effects of the aging that occur when the asphalt cement is heated to make asphalt concrete. The aged-residue grade numbers are at the middle of the allowable viscosity range after conditioning, as shown in Table 9.6 (ASTM D3381). For example, an AR-1000 has an absolute viscosity of 1000 ± 250 poises.

TABLE 9.6 Aged-Residue Grading System of Asphalt Cement

Grades	Viscosity		Penetration*	Flash Point* [@] °C (°F)
	Absolute (poises)	Kinematic* (cSt)		
AR-1000	1000 ± 250	140	65	205 (400)
AR-2000	2000 ± 500	200	40	219 (425)
AR-4000	4000 ± 1000	275	25	227 (440)
AR-8000	8000 ± 2000	400	20	232 (450)
AR-16000	16000 ± 4000	550	20	238 (460)

*Specification is for the minimum acceptable values

[@]Flash point specification is for the asphalt cement before rolling thin-film oven conditioning.
All other specifications are for samples that have been conditioned.

9.7.2 ■ Asphalt Cutbacks

Three types of cutbacks are produced, depending on the hardness of the residue and the type of solvent used. *Rapid-curing cutbacks* are produced by dissolving hard residue in a highly volatile solvent, such as gasoline. *Medium-curing cutbacks* use medium hardness residue and a less volatile solvent, such as kerosene. *Slow-curing cutbacks* are produced by either diluting soft residue in nonvolatile or low-volatility fuel oil or by simply stopping the refining process before all of the fuel oil is removed from the stock.

Curing the cutback refers to the evaporation of the solvent from the asphalt residue. Rapid-curing (RC) cutbacks cure in about 5 to 10 minutes, while medium-curing (MC) cutbacks cure in a few days. Slow-curing (SC) cutbacks cure in a few months. In addition to the three types, cutbacks have several grades defined by the kinematic viscosity at 60°C (140°F). Grades of 30, 70, 250, 800, and 3000 are manufactured, with higher grades indicating higher viscosities. Thus, cutback asphalts are designated by letters (RC, MC, or SC), representing the type, followed by a number that represents the grade. For example, MC-800 is a medium-curing cutback with a grade of 800. The different grades of cutback are produced by varying the amounts and types of solvent and base asphalt. The specifications of cutbacks are standardized by ASTM D2026, D2027, and D2028.

9.7.3 ■ Asphalt Emulsions

Asphalt emulsions are produced in a variety of combinations of the electric charge of the emulsifying agent, the rate the emulsion sets (brakes), the viscosity of the emulsion, and the hardness of the asphalt cement. Both

anionic and cationic are produced. These emulsions have negative and positive charges respectively. Depending on the emulsion concentration, the set or break time is varied from rapid to medium to slow set. Rapid-setting emulsion sets in about 5 to 10 minutes, medium-setting emulsion sets in several hours, and slow-setting emulsion sets in a few months. The viscosity of the emulsion is rated as normal flow or slow flow, based on the Saybolt–Furol viscosity (Figure 9.21). The consistency of the asphalt in the emulsion is evaluated with the penetration test. Asphalt residues with a penetration of 100–200 are typically used. However soft asphalts, with a penetration of greater than 200, or hard asphalts, with a penetration of 60–100, may be used.

Asphalt emulsion types are designated based on the setting rate as RS, MS or SS for rapid, medium, and slow set respectively. If a cationic emulsifying agent was used, the set designation is preceded by a C (e.g., a cationic rapid set emulsion is designated as CRS). If the first letter of the emulsion type is not a C, then it is an anionic emulsion. The flow rate of the emulsion is designated next with a 1 or 2 for a normal or slow flow respectively. Finally, the consistency of the asphalt is identified. Not all possible combinations of charge type, set rate, viscosity, and asphalt hardness are produced. Table 9.7 is a summary of the emulsion grades and types (Jansich and Gaillard, 1998).

Other emulsion types are also produced, such as the high float residue emulsion and the quick-set emulsion. The specifications of various asphalt emulsions are standardized by ASTM D977.

TABLE 9.7 Asphalt Emulsion Grades

Charge	Grade	Setting Speed	Viscosity of Emulsion, Saybolt Furol at 25°C	Penetration of Residue
Anionic	RS-1	Rapid	20–100	100–200
	RS-2	Rapid	75–400*	100–200
	MS-1	Medium	20–100	100–200
	MS-2	Medium	≥100	100–200
	MS-2h	Medium	≥100	60–100
	SS-1	Slow	20–100	100–200
	SS-1h	Slow	20–100	60–100
Cationic	CRS-1	Rapid	20–100	100–250
	CRS-2	Rapid	100–400	100–250
	CMS-2	Medium	50–450	100–250
	CMS-2h	Medium	50–450	60–100
	CSS-1	Slow	20–100	100–250
	CSS-1h	Slow	20–100	60–100

*Test at 50°C

9.8 Asphalt Concrete

Asphalt concrete, also known as hot-mix asphalt (HMA), consists of asphalt cement and aggregates mixed together at a high temperature and placed and compacted on the road while still hot. Asphalt (flexible) pavements cover approximately 93% of the 2 million miles of paved roads in the United States, while the remaining 7% of the roads are portland cement concrete (rigid) pavements. The performance of asphalt pavements is largely a function of the asphalt concrete surface material.

The objective of the asphalt concrete mix design process is to provide the following properties (Roberts et al. 1996):

1. stability or resistance to permanent deformation under the action of traffic loads, especially at high temperatures
2. fatigue resistance to prevent fatigue cracking under repeated loadings
3. resistance to thermal cracking that might occur due to contraction at low temperatures
4. resistance to hardening or aging during production in the mixing plant and in service
5. resistance to moisture-induced damage that might result in stripping of asphalt from aggregate particles
6. skid resistance, by providing enough texture at the pavement surface
7. workability, to reduce the effort needed during mixing, placing and compaction

Regardless of the set of criteria used to state the objectives of the mix design process, the design of asphalt concrete mixes requires compromises. For example, extremely high stability often is obtained at the expense of lower durability, and vice versa. Thus, in evaluating and adjusting a mix design for a particular use, the aggregate gradation and asphalt content must strike a favorable balance between the stability and durability requirements. Moreover, the produced mix must be practical and economical.

9.9 Asphalt Concrete Mix Design

The purpose of asphalt concrete mix design is to determine the design asphalt content using the available asphalt and aggregates. The design asphalt content varies for different material types, material properties, loading levels, and environmental conditions. To produce good-quality asphalt concrete, it is necessary to accurately control the design asphalt content. If the appropriate design asphalt content is not used, the pavement will lack durability or stability, resulting in premature pavement failure. For example, if not enough asphalt binder is used, not all the aggregate particles will be coated with asphalt, which will result in a less stable and less durable material. Also, if too

much binder is used, aggregate particles may have too much “lubrication” and may move relative to each other upon application of the load, resulting in a less stable material. Typical design asphalt contents range from 4% to 7% by weight of total mix.

Before the Superpave mix design method was developed during the SHRP program, there were two common asphalt concrete design methods: Marshall (ASTM D1559), and Hveem (ASTM D1560). The Marshall method was more commonly used than the Hveem method, due to its relative simplicity and its ability to be used for field control. Both methods are empirical in nature; that is, they are based on previous observations. Both methods have been used satisfactorily for several decades and have produced long-lasting pavement sections. However, due to their empirical nature they are not readily adaptable to new conditions, such as modified binders, large-sized aggregates, and heavier traffic loads.

The Superpave design system is performance based and is more rational than the Marshall and Hveem methods. Many highway agencies are implementing the Superpave system.

9.9.1 ■ Specimen Preparation in the Laboratory

Asphalt concrete specimens are prepared in the laboratory for mix-design and quality-control tests. To prepare specimens in the lab, aggregates are batched and heated, according to a specified gradation. Asphalt cement is also heated separately and added to the aggregate at a specified rate. Aggregates and asphalt are mixed with a mechanical mixer until the aggregate particles are completely coated with asphalt. Three compaction machines are commonly used:

1. Superpave gyratory compactor
2. Marshall hammer
3. California kneading compactor

Regardless of the compaction method, the procedure for preparing specimens basically follows the same four steps:

1. Heat and mix the aggregate and asphalt cement
2. Place the material into a mold
3. Apply compactive force
4. Allow the specimen to cool and extrude from the mold

The specific techniques for placing the material into the mold vary among the three compaction methods, and the standards for the test must be followed.

The greatest difference among the compaction procedures is the manner in which the compactive force is applied. For the gyratory compaction, the mixture in the mold is placed in the compaction machine at an angle to the applied force. As the force is applied the mold is gyrated, creating a shearing action in the mixture. Gyratory compaction devices have been available for a long time, but their use was limited due to the lack of a

mix-design procedure based on this type of compaction. However, the Superpave mix design method (FHWA, 1995) uses a gyratory compactor; thus, this compaction method is now common. Figure 9.23 shows the Superpave gyratory compactor.

In the Marshall procedure (Figure 9.24), a slide hammer weighing 4.45 kg (10 lb) is dropped from a height of 0.46 m (18 in.) to create an impact compaction force (ASTM D1559). The head of the Marshall hammer has a diameter equal to the specimen size, and the hammer is held flush with the specimen at all times.

In the California kneading compactor method (Figure 9.25), the area of the compactor foot is smaller than the area of the mold. After each compaction stroke the mold is rotated, subjecting the asphalt mixture to a kneading action (ASTM D1561). After the kneading compaction is complete, the specimen is

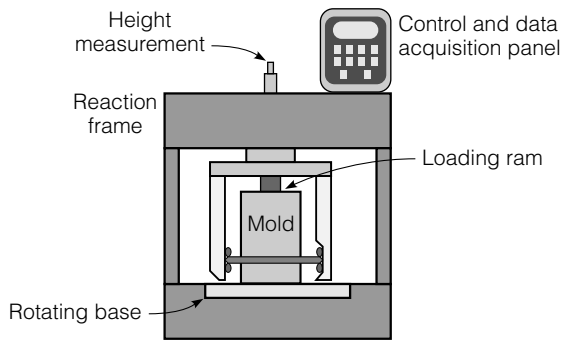


FIGURE 9.23 Superpave gyratory compactor.



FIGURE 9.24 Marshall compactor.



FIGURE 9.25 California kneading compactor.

reheated while still in the mold; then a compression machine is used to apply a static force to level the face of the specimen.

The Superpave gyratory compactor is used for the Superpave mix design, whereas the Marshall hammer and the California kneading compactor are used for the Marshall and Hveem methods of mix design, respectively. The Superpave gyratory compactor produces specimens 150 mm in diameter and 95 mm to 115 mm high, allowing the use of aggregates with a maximum size of more than 25 mm (1 in.). Specimens prepared with both Marshall and California kneading compactors, as well as some gyratory compactors, are typically 101.6 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high.

9.9.2 ■ Density and Voids Analysis

It is important to understand the density and voids analysis of compacted asphalt mixtures for both mix design and construction control. Regardless of the method used, the mix design is a process to determine the volume of asphalt binder and aggregates required to produce a mixture with the desired properties. However, since volumes are difficult and not practical to measure, weights are used instead; the specific gravity is used to convert from weight to volume. Figure 9.26 shows that the asphalt mixture consists of aggregates, asphalt binder, and air voids. Note that a portion of the asphalt is absorbed by aggregate particles. Three important parameters commonly used

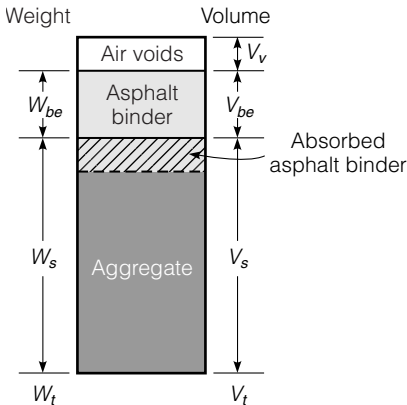


FIGURE 9.26 Components of compacted asphalt mixture.

are percent of air voids (voids in total mix) (VTM), voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA). These are defined as

$$VTM = \frac{V_v}{V_m} 100 \tag{9.1}$$

$$VMA = \frac{V_v + V_{be}}{V_m} 100 \tag{9.2}$$

$$VFA = \frac{V_{be}}{V_{be} + V_v} 100 \tag{9.3}$$

where

- V_v = volume of air voids
- V_{be} = volume of effective asphalt binder
- V_m = total volume of the mixture

The effective asphalt is the total asphalt minus the absorbed asphalt.

Sample Problem 9.2

A compacted asphalt concrete specimen contains 5% asphalt binder (Sp. Gr. 1.023) by weight of total mix, and aggregate with a specific gravity of 2.755. The bulk density of the specimen is 2.441 Mg/m³. Ignoring absorption, compute VTM, VMA, and VFA.

Solution

Referring to Figure SP 9.2, assume $V_t = 1 \text{ m}^3$
 Determine mass of mix and components:

- Total mass = $1 \times 2.441 = 2.441 \text{ Mg}$
- Mass of asphalt binder = $0.05 \times 2.441 = 0.122 \text{ Mg}$
- Mass of aggregate = $0.95 \times 2.441 = 2.319 \text{ Mg}$

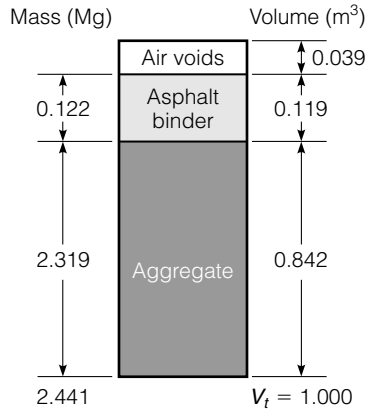


FIGURE SP9.2

Determine volume of components:

$$V_b = \frac{0.122}{1.023} = 0.119 \text{ m}^3$$

Since the problem statement specified no absorption, $V_{be} = V_b$

$$V_s = \frac{2.319}{2.755} = 0.842 \text{ m}^3$$

V_s = Volume of stone (aggregate)

Determine volume of voids:

$$V_v = V_m - V_b - V_s = 1 - 0.119 - 0.842 = 0.039 \text{ m}^3$$

Volumetric calculations:

$$\text{VTM} = \frac{V_v}{V_m} 100 = \frac{0.039}{1.00} 100 = 3.9\%$$

$$\text{VMA} = \frac{V_v + V_{be}}{V_m} 100 = \frac{0.039 + 0.119}{1.00} 100 = 15.8\%$$

$$\text{VFA} = \frac{V_{be}}{V_{be} + V_v} 100 = \frac{0.119}{0.119 + 0.039} 100 = 75\%$$

The density and void analysis requires using the effective specific gravity of the asphalt-coated aggregate, determined from the theoretical maximum specific gravity of the mix (ASTM D2041). The weight of the loose mixture specimen in air A is measured along with the weight of the measurement bowl filled with water D and the weight of the bowl containing the asphalt mix and filled with water E . When the loose mixture specimen is

submerged in water, a vacuum is used to remove all air from the sample. The theoretical maximum specific gravity is

$$G_{\text{mm}} = \frac{A}{A + D - E} \quad (9.4)$$

It is necessary to determine only the theoretical maximum specific gravity of the sample at one asphalt content. However, the result should be based on the average of three samples (with a minimum of two). By definition, the theoretical maximum specific gravity of asphalt concrete is

$$G_{\text{mm}} = \frac{100}{\left(\frac{P_s}{G_{\text{se}}} + \frac{P_b}{G_b}\right)} \quad (9.5)$$

Solving this equation for G_{se} produces

$$G_{\text{se}} = \frac{P_s}{\left(\frac{100}{G_{\text{mm}}} - \frac{P_b}{G_b}\right)} \quad (9.6)$$

where

- G_{mm} = theoretical maximum specific gravity of the asphalt concrete
- P_s = percent weight of the aggregate
- P_b = percent weight of the asphalt cement
- G_{se} = effective specific gravity of aggregate coated with asphalt
- G_b = specific gravity of the asphalt binder

Although G_{se} is determined for only one asphalt content, it is constant for all asphalt contents. Thus, once G_{se} is determined based on the results of the theoretical maximum specific gravity test, it can be used in Equation 9.5 to calculate G_{mm} for the different asphalt contents.

The next step in the process is to determine the bulk specific gravity G_{mb} (ASTM D2726) of each of the compacted specimens. This requires weighing the sample in three conditions: dry, saturated-surface dry, and submerged. The bulk specific gravity is computed as

$$G_{\text{mb}} = \frac{\text{Weight in air}}{(\text{Weight SSD} - \text{Weight in water})} \quad (9.7)$$

The unit weight of each specimen is computed by multiplying the bulk specific gravity by the density of water, 1 Mg/m³ (62.4 lb/ft³). The average bulk specific gravity and unit weight for each asphalt content are computed and used to calculate VTM as follows:

$$\text{VTM} = 100 \left(1 - \frac{G_{\text{mb}}}{G_{\text{mm}}} \right) \quad (9.8)$$

The percent voids in the mineral aggregate (VMA), is a measure of the space available in the aggregates for the addition of the asphalt cement. The percent VMA is the volume of the mix minus the volume of the aggregates, divided by the volume of the mix and converted to a percent. VMA is commonly computed from the bulk specific gravity of the aggregate G_{sb} , the bulk specific gravity of the mix G_{mb} and the percent weight of aggregate as

$$\text{VMA} = \left(100 - G_{mb} \frac{P_s}{G_{sb}} \right) \quad (9.9)$$

The percent of the voids filled with asphalt, %VFA, is determined as

$$\text{VFA} = 100 \left(\frac{\text{VMA} - \text{VTM}}{\text{VMA}} \right) \quad (9.10)$$

Sample Problem 9.3

An asphalt concrete specimen has the following properties:

- asphalt content = 5.9% by total weight of mix
- bulk specific gravity of the mix = 2.457
- theoretical maximum specific gravity = 2.598
- bulk specific gravity of aggregate = 2.692

Calculate the percents VTM, VMA, and VFA.

Solution

$$\text{VTM} = 100 \left(1 - \frac{G_{mb}}{G_{mm}} \right) = 100 \left(1 - \frac{2.457}{2.598} \right) = 5.4\%$$

$$\text{VMA} = \left(100 - G_{mb} \frac{P_s}{G_{mm}} \right) = \left(100 - 2.457 \frac{100 - 5.9}{2.692} \right) = 14.1\%$$

$$\text{VFA} = 100 \left(\frac{\text{VMA} - \text{VTM}}{\text{VMA}} \right) = 100 \left(\frac{14.1 - 5.4}{14.1} \right) = 61.7\%$$

9.9.3 Superpave Mix Design

The Superpave mix-design process consists of

- Selection of aggregates
- Selection of binder
- Determination of the design aggregate structure
- Determination of the design binder content.
- Evaluation of moisture susceptibility

Aggregate Selection Aggregate properties under the Superpave mix-design system are described as either consensus or source properties. The following *consensus* aggregate properties are required:

- coarse aggregate angularity measured by the percentage of fractured faces
- fine aggregate angularity (AASHTO TP 33) (see apparatus in Figure 5.5)
- flat and elongated particles (ASTM D4791)
- clay content (ASTM D2419)

Specification limits for these properties depend on the traffic level and how deep under the pavement surface the materials will be used, as shown in Table 9.8. In addition to these properties, highway agencies may consider other factors that are critical to the specific local conditions. These are called *source* properties and may include Los Angeles abrasion (see apparatus in Figure 5.6), soundness, and deleterious materials. Source properties are defined at the local level; consensus properties are defined at the national level.

Aggregate used in asphalt concrete must be well graded. Superpave recommends using the 0.45 power chart discussed in Chapter 5. The gradation curve should go through control points specified by Superpave. Figure 9.27 shows the gradation requirements for the 12.5 mm (1/2 in.) nominal-sized mix.

To control segregation, aggregates are sorted into stockpiles based on size. The designer of an asphalt concrete mix must select a blend of stockpiles that meets the source, consensus, and gradation requirements.

Binder Selection The binder is selected based on the maximum and minimum pavement temperatures, as discussed earlier. In addition to the specification tests, the specific gravity and the rotational viscosity versus temperature relationship for the selected asphalt binder must be measured. The specific gravity is needed for the void analysis. The viscosity–temperature relationship is needed to determine the required mixing and compaction temperatures. The Superpave method requires mixing the asphalt and aggregates at a temperature at which the rotational viscosity of the asphalt binder is $0.170 \pm 20 \text{ Pa} \cdot \text{s}$ and the compacting temperature corresponds to a viscosity of $0.280 \pm 30 \text{ Pa} \cdot \text{s}$.

TABLE 9.8 Superpave Consensus Aggregate Properties

Design Level	Course Aggregate Angularity (% min)	Fine Aggregate Angularity (% min)	Sand Equivalency (% min)	Flat and Elongated (% min)
Light Traffic	55/-	—	40	—
Medium Traffic	75/-	40	40	10
Heavy Traffic	85/80*	45	45	10

*85/80 denotes minimum percentages of one fractured face / two or more fractured faces

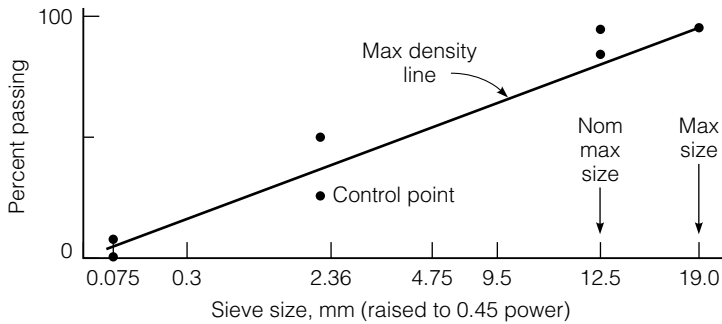


FIGURE 9.27 Superpave gradation limits for 12.5 mm nominal maximum size.

Design Aggregate Structure After selecting the appropriate aggregate, binder, and modifiers (if any), trial specimens are prepared with three different aggregate gradations and asphalt contents. There are equations for estimating the optimum asphalt content for use in the specimens prepared for determining the design aggregate structure. However, these equations are empirical and the designer is given latitude to estimate the asphalt content. Specimens are compacted using the Superpave gyratory compactor (Figure 9.23) with a gyration angle of 1.25 degrees and a constant vertical pressure of 600 kPa (87 psi). The number of gyrations used for compaction is determined based on the traffic level, as shown in Table 9.9.

As shown in Table 9.9, the Superpave method recognizes three critical stages of compaction, initial, design, and maximum. The design compaction level N_{des} corresponds to the compaction that is anticipated at the completion of the construction process. The maximum compaction N_{max} corresponds to the ultimate density level that will occur after the pavement has

TABLE 9.9 Number of Gyration at Specific Design Traffic Levels

	Traffic Level (10^6 ESAL*)			
	<0.3	0.3 to 3	3 to 30	>30
N_{ini}	6	7	8	9
N_{des}	50	75	115	125
N_{max}	75	100	160	205

*ESAL is the 18,000-lb equivalent single axle load. It is a design factor used in the design of pavement that considers both traffic volume and loads (Huang 2004).

been subjected to traffic for a number of years. The initial compaction level N_{ini} was implemented to assist with identifying “tender” mixes. A tender mix lacks stability during construction, and hence will displace under the rollers rather than densifying. For the initial stage of determining the design aggregate structure, samples are compacted with N_{des} gyrations. The volumetric properties are determined by measuring the bulk specific gravity G_{mb} of the compacted mix and the maximum theoretical specific gravity G_{mm} of a loose mix with the same asphalt content and aggregate composition. The volumetric parameters, VTM, VMA, and VFA, are determined and checked against the criteria. Two additional parameters are evaluated in the Superpave method: percent of G_{mm} at N_{ini} , and the dust-to-effective binder content ratio. The percent of G_{mm} at N_{ini} is determined as

$$\text{Percent } G_{mm,Nini} = \text{Percent } G_{mm,Ndes} \frac{h_{des}}{h_{ini}} \quad (9.11)$$

where h_{ini} and h_{des} are the heights of the specimen at the initial and design number of gyrations, respectively. Note that the percent $G_{mm,Ndes}$ is equal to (100-VTM).

The dust-to-effective binder ratio is the percent of aggregate passing the 0.075 mm (#200) sieve divided by the effective asphalt content, computed as

$$P_{ba} = 100 \left(\frac{G_{se} - G_{sb}}{G_{sb}G_{se}} \right) G_b \quad (9.12)$$

$$P_{be} = P_b - \left(\frac{P_{ba}}{100} \right) P_s \quad (9.13)$$

$$D/B = \frac{P_D}{P_{be}} \quad (9.14)$$

where

D/B = dust to binder ratio

P_{ba} = percent absorbed binder based on the mass of aggregates

G_{sb} = bulk specific gravity of aggregate

P_D = percent dust, or % of aggregate passing the 0.075mm sieve

P_{be} = percent effective binder content

The Superpave method requires determining the volumetric properties at 4% VTM. The samples prepared for the evaluation of the design aggregate content are not necessarily at the binder content required for achieving this level of air voids. Therefore, the results of the volumetric evaluation are “corrected” to four percent air voids, as

$$P_{b,est} = P_{bt} - 0.4(4 - VTM_t) \quad (9.15)$$

$$VMA_{est} = VMA_t + C(4 - VTM_t) \quad (9.16)$$

$$C = 0.1 \text{ for } VTM_t < 4.0\%$$

$$C = 0.2 \text{ for } VTM_t \geq 4.0\%$$

$$VFA_{est} = 100 \frac{VMA_t - 4.0}{VMA_t} \quad (9.17)$$

TABLE 9.10 Superpave Mix Design Criteria						
Design Air Voids	4%					
Dust to Effective Asphalt ¹	0.6–1.2					
Tensile strength ratio	80% min					
	Nominal Maximum Size (mm)					
	37.5	25	19	12.5	9.5	4.75
Minimum VMA (%)	11	12	13	14	15	16
G_{mm} and VFA Requirements						
Design EASL in millions	Percent Maximum Theoretical Specific Gravity			Percent Voids Filled with Asphalt ^{2,3,4}		
	N_{init}	N_{des}	N_{max}			
<0.3	≤91.5	96	≤98.0	70–80		
0.3–3	≤90.5	96	≤98.0	65–78		
3–10	≤89.0	96	≤98.0	65–75		
10–30	≤89.0	96	≤98.0	65–75		
≥30	≤89.0	96	≤98.0	65–75		

Notes

1. Dust-to-binder ratio range is 0.9 to 2.0 for 4.75 mm mixes.
2. For 9.5 mm nominal maximum aggregate size mixes and design VFA ≥ 3 million, VFA range is 73 to 76% and for 4.75 mm mixes the range is 75 to 78%.
3. For 25 mm nominal maximum aggregate size mixes, the lower limit of the VFA range shall be 67% for design traffic levels <0.3 million ESALs.
4. For 37.5 mm nominal maximum aggregate size mixes, the lower limit of the VFA range shall be 64% for all design traffic levels.

$$P_{b,est} = P_{b,est} - \frac{P_s G_b (G_{se} - G_{sb})}{G_{se} G_{sb}} \quad (9.18)$$

$$D/B_{est} = \frac{P_D}{P_{b,est}} \quad (9.19)$$

where

- $P_{b,est}$ = adjusted estimated binder content
- VMA_{est} = adjusted VMA
- VMA_t = VMA determined from volumetric analysis
- VFA_{est} = adjusted VFA

VTM_t = VTM determined from the volumetric analysis

$P_{be,est}$ = adjusted percent effective binder

D/B_{est} = adjusted dust-to-binder ratio

P_D = percent aggregate passing 0.075mm sieve

In Equation 9.17, the air voids are assumed to be 4.0%, the target air void content for a Superpave mix design.

The adjusted results of the design aggregate structure evaluation are compared to the Superpave mix-design criteria, as shown in Table 9.10.

The design aggregate blend is the one whose adjusted volumetric parameters meet all of the criteria. In the event that more than one of the blends meets all of the criteria, the designer can use discretion for the selection of the blend. A pair of samples is compacted at the adjusted binder content for the selected aggregate blend. The average percent of maximum theoretical specific gravity is determined and compared to the design criteria. If successful, the procedure continues with the determination of the design binder content. If unsuccessful, the design process is started over with the selection of another blend of aggregates.

Sample Problem 9.4

Select a 19-mm Superpave design aggregate structure based on the following data:

Data	Blend		
	1	2	3
G_{mb}	2.457	2.441	2.477
G_{mm}	2.598	2.558	2.664
G_b	1.025	1.025	1.025
P_b (%)	5.9	5.7	6.2
P_s (%)	94.1	94.3	93.8
P_d (%)	4.5	4.5	4.5
G_{sb}	2.692	2.688	2.665
h_{ini} (mm)	125	131	125
h_{des} (mm)	115	118	115

Solution

In this problem, the lab data were entered into an Excel spreadsheet and the appropriate equations were entered, producing the results shown in the accompanying table.

The steps in the calculation are shown for the first blend only; the reader can verify the calculations for the other blends

Volumetric Analysis		Blend			
Computed	Equation	Using Data for Blend 1	1	2	3
G_{se}	9.6	$\frac{P_s}{\left(\frac{100}{G_{mm}} - \frac{P_b}{G_b}\right)}$	2.875	2.812	2.979
VTM (%)	9.8	$100 \left(1 - \frac{G_{mb}}{G_{mm}}\right)$	5.4	4.6	7.0
VMA (%)	9.9	$\left(100 - G_{mb} \frac{P_s}{G_{sb}}\right)$	14.1	14.4	12.8
VFA (%)	9.10	$100 \left(\frac{VMA - VTM}{VMA}\right)$	61.7	68.1	45.3
% $G_{mm,Nini}$	9.11	$\text{Percent } G_{mm,Ndes} \frac{h_{des}}{h_{ni}}$	87.0	86.0	85.5
P_{ba} (%)	9.12	$100 \left(\frac{G_{se} - G_{sb}}{G_{sb} G_{se}}\right) G_b$	1.025	1.68	4.05
P_{be} (%)	9.13	$P_b - \left(\frac{P_{ba}}{100}\right) P_s$	3.62	4.12	2.4
D/B	9.14	$\frac{P_D}{P_{be}}$	1.2	1.1	1.9
Adjusted Values		Criteria			
$P_{b,est}$ (%)	9.15	$P_{bt} - 0.4(4 - VTM_t)$	6.5	5.9	7.4
VMA _{est} (%)	9.16	$VMA_t + C(4 - VTM_t)$	13.8	14.3	12.2
VFA _{est} (%)	9.17	$100 \frac{VMA_t - 4.0}{VMA_t}$	71.0	72.0	65-75
$P_{be,est}$ (%)	9.18	$P_{b,est} - \frac{P_s G_b (G_{se} - G_{sb})}{G_{se} G_{sb}}$	4.2	4.3	3.6
D/B _{est}	9.19	$\frac{P_D}{P_{be,est}}$	1.1	1.0	1.3

Although both mixes 1 and 2 meet the criteria, blend 2 is preferred, since it has a higher VMA value.

Design Binder Content The design binder content is obtained by preparing eight specimens—two replicates at each of four binder contents: estimated optimum binder content, 0.5% less than the optimum, 0.5% more than the optimum, and 1% more than the optimum. Specimens are compacted using N_{des} gyrations. The volumetric properties are computed and plots are prepared of each volumetric parameter versus the binder content. The binder content that corresponds to 4% VTM is determined. Then the plots are used to determine the volumetric parameters at the selected binder content. If the properties meet the criteria in Table 9.10, two specimens are prepared and compacted using N_{max} gyrations. If these specimens meet the criteria, then the moisture sensitivity of the mix is determined.

Sample Problem 9.5

Based on the previous problem, determine the recommended asphalt content according to Superpave mix design for an equivalent single axle load (ESAL) of 20 millions.

Solution

Samples were prepared at four asphalt contents. An Excel spreadsheet was prepared to present and analyze the data. The following results are obtained:

The results are plotted against binder content, as shown in Figure SP9.5.

Design Binder Content

Data	Asphalt Content Trial			
	1	2	3	4
P_b (%)	5.4	5.9	6.4	6.9
G_{mb}	2.351	2.441	2.455	2.469
G_{mm}	2.570	2.558	2.530	2.510
G_b	1.025	1.025	1.025	1.025
P_s (%)	94.6	94.1	93.6	93.1
P_d (%)	4.5	4.5	4.5	4.5
G_{sb}	2.688	2.688	2.688	2.688
h_{ini} (mm)	125	131	126	130
h_{des} (mm)	115	118	114	112

Design Binder Content

Volumetric Analysis

Computed	Equation	Asphalt Content Trial			
		1	2	3	4
G_{se}	9.6	2.812	2.812	2.812	2.812
VTM (%)	9.8	8.5	4.6	3.0	1.6
VMA (%)	9.9	17.3	14.5	14.5	14.5
VFA (%)	9.10	50.9	68.3	79.3	89.0
% G_{mm}, N_{ini}	9.11	84.2	86.0	87.8	84.7
P_{ba} (%)	9.12	1.68	1.68	1.68	1.68
P_{be} (%)	9.13	3.81	4.32	4.83	5.34
D/B	9.14	1.2	1.0	0.9	0.8

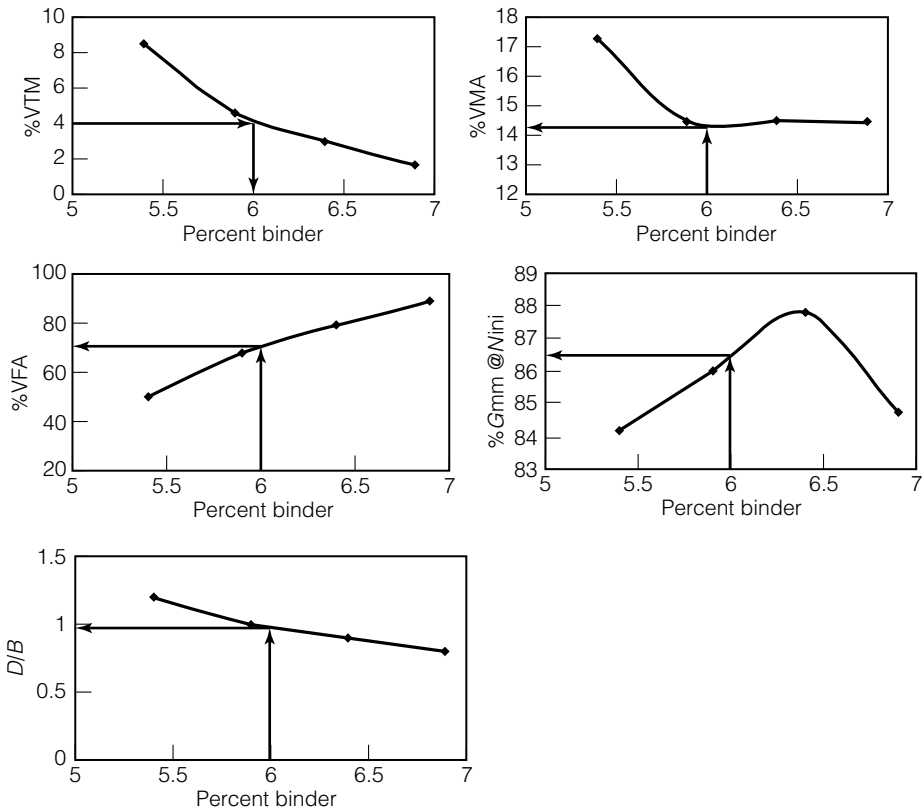


FIGURE SP9.5

The figure shows that, at 4% VTM, the binder content is 6%. The design values at 6% binder content are

- VMA = 14.5%
- VFA = 71%
- $G_{mm} @ N_{ini} = 86.5\%$
- D/B ratio = 0.9

These values satisfy the design criteria shown in Table 9.10. Therefore, the design binder content is 6.0%.

Moisture Sensitivity Evaluation The moisture sensitivity of the design mixture is determined using the AASHTO T283 procedure on six specimens prepared at the design binder content and 7% air voids. Three specimens are conditioned by vacuum saturation, then freezing and thawing; three other specimens are not conditioned. The tensile strength of each sample is measured using a Marshall stability machine with a modified loading head. The tensile strength ratio is determined as the ratio of the average tensile strength of conditioned specimens to that of unconditioned specimens. The minimum Superpave criterion for tensile strength ratio is 80%.

9.9.4 ■ Superpave Simple Performance Tests (SPT)

The volumetric procedure has been widely implemented with successful results. However, the method lacks a strength test to verify the suitability of the Superpave mixes. Research completed in NCHRP Project 9-19 has identified three candidate simple performance tests (SPT) for HMA (Witczak et al. 2002). Tests based on measurement of dynamic modulus (for both of permanent deformation and fatigue cracking), flow time (permanent deformation), and flow number (permanent deformation) were selected for further field validation. The three tests used to obtain these parameters are the dynamic modulus test, triaxial static creep test, and triaxial repeated load permanent deformation test. All these tests use cylindrical specimens 100 mm (4 in.) in diameter and 150 mm (6 in.) high. Specimens are cored in the lab from specimens compacted using the Superpave gyratory compactor (Figure 9.23). Figure 9.28 shows a photo of the SPT.

Dynamic Modulus Test The dynamic modulus test in triaxial compression has been around the pavement community for many years (ASTM D3497). The test consists of applying an axial sinusoidal compressive stress to an unconfined or confined HMA cylindrical test specimen, as shown in Figure 9.29.

Assuming that HMA is a linear viscoelastic material, a complex number called the complex modulus, E^* , can be obtained from the test to define the relationship between stress and strain. The absolute value of the complex modulus, $|E^*|$, is called the dynamic modulus. The dynamic modulus is mathematically defined as the peak dynamic stress σ_o divided by the peak recoverable axial strain ϵ_o :

$$|E^*| = \frac{\sigma_o}{\epsilon_o} \quad (9.20)$$



FIGURE 9.28 Superpave simple performance test assembly.

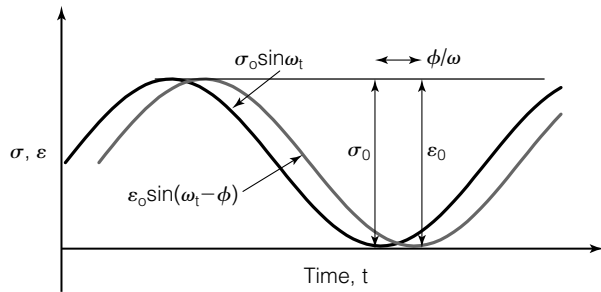


FIGURE 9.29 Stress and strain pulses for the dynamic modulus test.

The real and imaginary portions of the complex modulus E^* can be written as

$$E^* = E' + iE'' \tag{9.21}$$

E' is generally referred to as the storage or elastic modulus component of the complex modulus and E'' is referred to as the loss or viscous modulus. The phase angle ϕ is the angle by which ϵ_o lags behind σ_o . It is an indicator

of the viscous properties of the material being evaluated. Mathematically, this is expressed as

$$E^* = |E^*| \cos \phi + i|E^*| \sin \phi \tag{9.22}$$

where

$$\phi = \frac{t_i}{t_p} \times 360$$

t_i = time lag between of stress and strain, s

t_p = time for a stress cycle, s

i = imaginary number

For a pure elastic material, $\phi = 0$, and the complex modulus E^* is equal to the absolute value, or the dynamic modulus. For a pure viscous material, $\phi = 90^\circ$.

The dynamic modulus obtained from this test is indicative of the stiffness of the asphalt mixture at the selected temperature and load frequency. The dynamic modulus is correlated to both rutting and fatigue cracking of HMA.

Triaxial Static Creep Test In the static compressive creep test, a total strain–time relationship for a mixture is measured in the laboratory under unconfined or confined conditions. The static creep test, using either one load–unload cycle or incremental load–unload cycles, provides sufficient information to determine the instantaneous elastic (recoverable) and plastic (irrecoverable) components (time independent), and the viscoelastic and viscoplastic components (time dependent) of the material’s response. In this test, the compliance, $D(t)$ is determined by dividing the strain as a function of time $\varepsilon(t)$, by the applied constant stress σ_o :

$$D(t) = \frac{\varepsilon(t)}{\sigma_o} \tag{9.23}$$

Figure 9.30 shows typical test results between the calculated compliance and loading time. As shown, the compliance can be divided into three

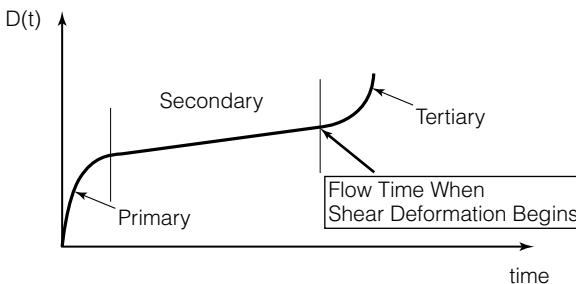


FIGURE 9.30 Compliance versus loading time for the static triaxial creep test.

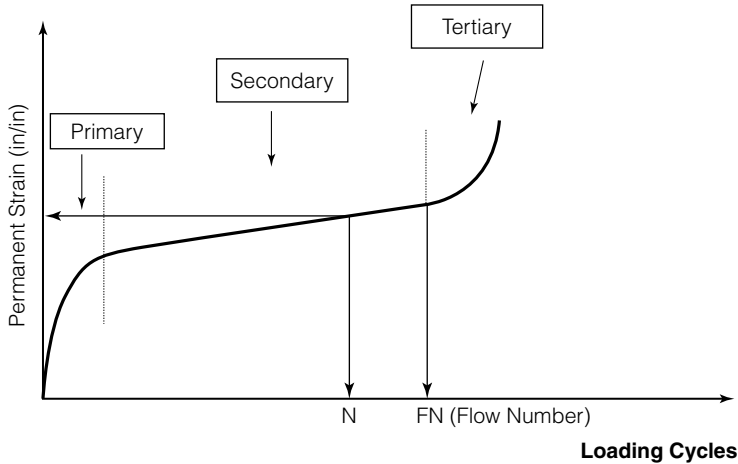


FIGURE 9.31 Typical relationship between total cumulative plastic strain and loading cycles.

major zones: primary zone, secondary zone, and tertiary flow zone. The time at which tertiary flow starts is referred to as the *Flow Time*. The flow time is a significant parameter in evaluating the rutting resistance of HMA.

Triaxial Repeated Load Permanent Deformation Test Another approach to measuring the permanent deformation characteristics of HMA is to use a repeated load test for several thousand repetitions and to record the cumulative permanent deformation as a function of the number of load repetitions. In this test, a haversine pulse load consisting of a 0.1 second and 0.9 second dwell (rest) time is applied for the test duration, typically about three hours or 10,000 loading cycles.

Results from the repeated load tests typically are presented in terms of the cumulative permanent strain versus the number of loading cycles. Figure 9.31 illustrates a typical relationship between the cumulative plastic strain and number of load cycles. In a manner similar to the triaxial static creep test, the cumulative permanent strain curve can be divided into three zones: primary, secondary, and tertiary. The cycle number at which tertiary flow starts is referred to as the *Flow Number*. In addition to the flow number, the test can provide the resilient strain and modulus, all of which are correlated to the rutting resistance of HMA.

9.9.5 ■ Marshall Method of Mix Design

The basic steps required for performing Marshall mix design are as follows (The Asphalt Institute 1995):

1. aggregate evaluation
2. asphalt cement evaluation

3. specimen preparation
 4. Marshall stability and flow measurement
 5. density and voids analysis
 6. design asphalt content determination
1. **Aggregate Evaluation** The aggregate characteristics that must be evaluated before it can be used for an asphalt concrete mix include the durability, soundness, presence of deleterious substances, polishing, shape, and texture. Agency specifications define the allowable ranges for aggregate gradation. The Marshall method is applicable to densely graded aggregates with a maximum size of not more than 25 mm (1 in.).
 2. **Asphalt Cement Evaluation** The grade of asphalt cement is selected based on the expected temperature range and traffic conditions. Most highway agencies have specifications that prescribe the grade of asphalt for the design conditions.
 3. **Specimen Preparation** The full Marshall mix-design procedure requires 18 specimens 101.6 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high. The stability and flow are measured for 15 specimens. In addition, 3 specimens are used to determine the theoretical maximum specific gravity G_{mm} . This value is needed for the void and density analysis. The specimens for the theoretical maximum specific gravity determination are prepared at the estimated design asphalt content. Samples are also required for each of five different asphalt contents; the expected design asphalt content, $\pm 0.5\%$ and $\pm 1.0\%$. Engineers use experience and judgment to estimate the design asphalt content.

Specimen preparation for the Marshall method uses the Marshall compactor discussed earlier (Figure 9.24). The Marshall method requires



FIGURE 9.32 Marshall stability machine.

mixing of the asphalt and aggregates at a temperature where the kinematic viscosity of the asphalt cement is 170 ± 20 cSt and compacting temperature corresponds to a viscosity of 280 ± 30 cSt.

The Asphalt Institute permits three different levels of energy to be used for the preparation of the specimens: 35, 50, and 75 blows on each side of the sample. Most mix designs for heavy-duty pavements use 75 blows, since this better simulates the required density for pavement construction.

4. **Marshall Stability and Flow Measurement** The Marshall stability of the asphalt concrete is the maximum load the material can carry when tested in the Marshall apparatus, Figure 9.32. The test is performed at a deformation rate of 51 mm/min. (2 in./min.) and a temperature of 60°C (140° F). The Marshall flow is the deformation of the specimen when the load starts to decrease. Stability is reported in newtons (pounds) and flow is reported in units of 0.25 mm (0.01 in.) of deformation. The stability of specimens that are not 63.5 mm thick is adjusted by multiplying by the factors shown in Table 9.11. All specimens are tested and the average stability and flow are determined for each asphalt content.
5. **Density and Voids Analysis** The values of VTM, VMA, and VFA are determined as using Equations 9.8, 9.9, and 9.10, respectively.
6. **Design Asphalt Content Determination** Traditionally, test results and calculations are tabulated and graphed to help determine the factors that must be used in choosing the optimum asphalt content. Table 9.12 presents examples of mix design measurements and calculations. Figure 9.33 shows plots of results obtained from Table 9.12, which include asphalt content versus air voids, VMA, VFA, unit weight, Marshall stability, and Marshall flow.

TABLE 9.11 Marshall Stability Adjustment Factors

Approximate Thickness of Specimen, mm (in.)	Adjustment Factor	Approximate Thickness of Specimen, mm (in.)	Adjustment Factor
50.8 (2)	1.47	65.1 (2 9/16)	0.96
52.4 (2 1/16)	1.39	66.7 (2 5/8)	0.93
54.0 (2 1/8)	1.32	68.3 (2 11/16)	0.89
55.6 (2 3/16)	1.25	69.8 (2 3/4)	0.86
57.2 (2 1/4)	1.19	71.4 (2 13/16)	0.83
58.7 (2 5/16)	1.14	73.0 (2 7/8)	0.81
60.3 (2 3/8)	1.09	74.6 (2 15/16)	0.78
61.9 (2 7/16)	1.04	76.2 (3)	0.76
63.5 (2 1/2)	1.00		

TABLE 9.12 Examples of Mix Design Measurements and Calculations by the Marshall Method (The Asphalt Institute, 1995)

% AC by Wt. of Mix, Spec. No.	Spec. Height, mm	Wt. in Air, g	Wt. in Water, g	SSD Wt., g	Bulk Vol., cm ³	Bulk Sp. Gr.	Max. Theo. Sp. Gr. (loose mix)	% Air Voids	% VMA	% VFA	Measured Stability, kN	Adjusted Stability, kN	Flow, 0.25 mm
3.5-A		1240.6	726.4	1246.3	519.9	2.386					10.9	10.9	8
3.5-B		1238.7	723.3	1242.6	519.3	2.385					10.8	10.8	7
3.5-C		1240.1	724.1	1245.9	521.8	2.377					11.2	11.2	7
Average						2.383	2.570	7.3	14.0	48.0		10.9	7
4.0-A		1244.3	727.2	1246.6	519.4	2.396					9.7	9.7	9
4.0-B		1244.6	727.0	1247.6	520.6	2.391					10.1	10.1	9
4.0-C		1242.6	727.9	1244.0	516.1	2.408					10.3	10.3	8
Average						2.398	2.550	6.0	13.9	57.1		10.0	9
4.5-A		1249.3	735.8	1250.2	414.4	2.429					10.8	10.8	9
4.5-B		1250.8	728.1	1251.6	523.5	2.389					10.7	10.3	9
4.5-C		1251.6	735.3	1253.1	517.8	2.417					10.4	10.4	9
Average						2.412	2.531	4.7	13.9	66.1		10.5	9
5.0-A		1256.7	739.8	1257.6	517.8	2.427					10.2	10.2	9
5.0-B		1258.7	742.7	1259.3	516.6	2.437					9.7	9.7	8
5.0-C		1258.4	737.5	1259.1	521.6	2.413					10.0	10.0	9
Average						2.425	2.511	3.4	13.8	75.2		10.0	9
5.5-A		1263.8	742.6	1264.3	521.7	2.422					9.8	9.8	9
5.5-B		1258.8	741.4	1259.4	518.0	2.430					10.2	10.2	10
5.5-C		1259.0	742.5	1259.5	517.0	2.435					9.8	10.0	9
Average						2.429	2.493	2.5	14.1	82.1		10.0	9

Notes: AC-20 binder, $G_b = 1.030$, $G_{sb} = 2.674$, Absorbed AC of aggregate: 0.6%, $G_{se} = 2.717$, Compaction: 75 blows

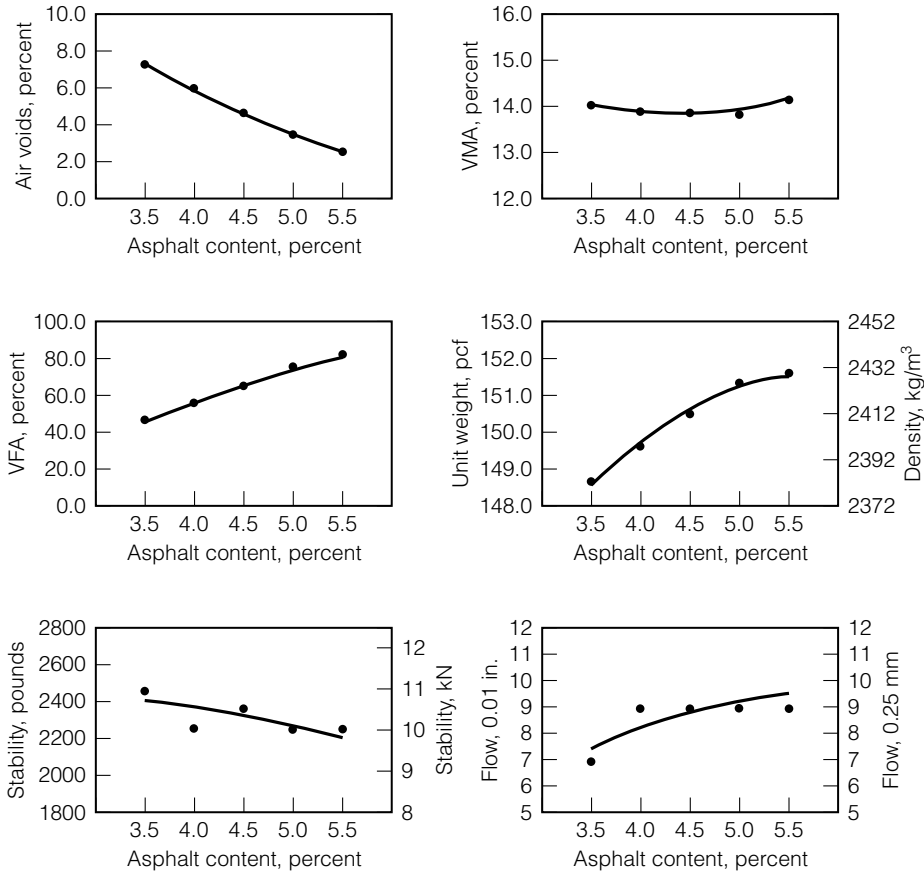


FIGURE 9.33 Graphs used for Marshall mix design analysis. (See Table 9.12) (The Asphalt Institute 1995).

The design asphalt content is usually the most economical one that will satisfactorily meet all of the established criteria. Different criteria are used by different agencies. Table 9.13 and 9.14 depict the mix design criteria recommended by The Asphalt Institute. Figure 9.34 shows an example of the narrow range of acceptable asphalt contents. The asphalt content selection can be adjusted within this narrow range to achieve a mix that satisfies the requirements of a specific project. Other agencies, such as the National Asphalt Paving Association, use the asphalt cement content at 4% air voids as the design value, and then check that the other factors meet the criteria. If the Marshall stability, Marshall flow, VMA, or VFA fall outside the allowable range, the mix must be redesigned using an adjusted aggregate gradation or new material sources.

The laboratory-developed mixture design forms the basis for the initial job mix formula (JMF). The initial JMF should be adjusted to reflect the slight differences between the laboratory-supplied aggregates and those used in the field.

TABLE 9.13 Asphalt Institute Criteria for Marshall Mix Design (The Asphalt Institute, 1995)

	Traffic Level					
	Light		Medium		Heavy	
Compaction (blows)	35		50		75	
	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
Stability, kN	3.34	—	5.34	—	8.01	—
Flow, 0.25 mm	8	18	8	16	8	14
Air Voids, %	3	5	3	5	3	5
VMA, %	Use the criteria in Table 9.14					
VFA, %	70	80	65	78	65	75

TABLE 9.14 Minimum Percent Voids in Mineral Aggregate (VMA) (The Asphalt Institute, 1995)

Nominal Maximum Particle Size ¹	Minimum VMA, Percent		
	Design Air Voids ²		
	3.0	4.0	5.0
2.36 mm (No. 8)	19.0	20.0	21.0
4.75 mm (No. 4)	16.0	17.0	18.0
9.5 mm (3/8 in.)	14.0	15.0	16.0
12.5 mm (1/2 in.)	13.0	14.0	15.0
19.0 mm (3/4 in.)	12.0	13.0	14.0
25.0 mm (1.0 in.)	11.0	12.0	13.0

¹The nominal maximum particle size is one size larger than the first sieve to retain more than 10 percent.

²Interpolate minimum VMA for design air void values between those listed.

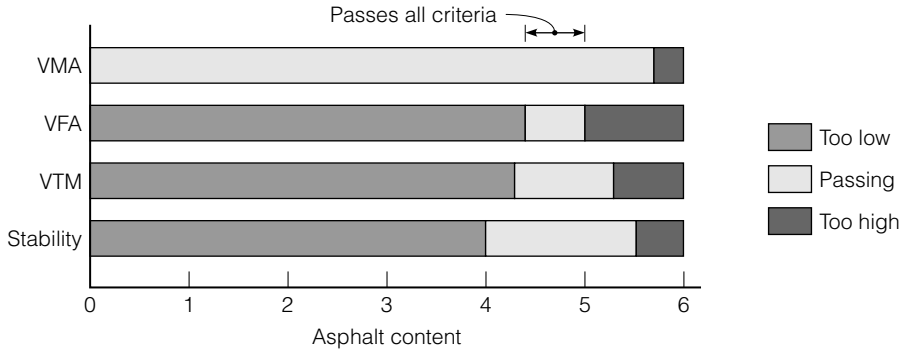


FIGURE 9.34 An example of the narrow range of acceptable asphalt contents. (The Asphalt Institute 1995).

Sample Problem 9.6

The Marshall method was used to design an asphalt concrete mixture. An AC-30 asphalt cement with a specific gravity (G_b) of 1.031 was used. The mixture contains a 9.5 mm nominal maximum particle size aggregate with a bulk specific gravity (G_{sb}) of 2.696. The theoretical maximum specific gravity of the mix (G_{mm}) at asphalt content of 5.0% is 2.470. Trial mixes were made with average results as shown in the following table:

Asphalt Content (P_b) (% by Weight of Mix)	Bulk Specific Gravity (G_{mb})	Corrected Stability (kN)	Flow (0.25 mm)
4.0	2.360	6.3	9
4.5	2.378	6.7	10
5.0	2.395	5.4	12
5.5	2.405	5.1	15
6.0	2.415	4.7	22

Determine the design asphalt content using the Asphalt Institute design criteria for medium traffic. (Table 9.13). Assume a design air void content of 4% when using Table 9.14.

Solution

Analysis steps:

1. Determine the effective specific gravity of the aggregates G_{se} , using Equation 9.6:

$$G_{se} = \frac{P_s}{\left(\frac{100}{G_{mm}} - \frac{P_b}{G_b}\right)} = \frac{100 - 5.0}{\left(\frac{100}{2.470} - \frac{5.0}{1.031}\right)} = 2.666$$

The calculations in steps 2–5 are for 4.0% asphalt content as an example. Repeat for other asphalt contents.

- Use G_{se} to determine G_{mm} for the other asphalt contents, using Equation 9.5:

$$G_{mm} = \frac{100}{\left(\frac{P_s}{G_{se}} + \frac{P_b}{G_b}\right)} = \frac{100}{\left(\frac{100 - 4.0}{2.666} + \frac{4.0}{1.031}\right)} = 2.507$$

- Compute voids in the total mix for each asphalt content, using Equation 9.8:

$$VTM = 100\left(1 - \frac{G_{mb}}{G_{mm}}\right) = 100\left(1 - \frac{2.360}{2.507}\right) = 5.9$$

- Compute voids in mineral aggregate, using Equation 9.9:

$$VMA = 100 - \left(\frac{G_{mb}P_s}{G_{mm}}\right) = 100 - \left(\frac{2.360 \times (100 - 4.0)}{2.666}\right) = 16.0$$

- Compute voids filled with asphalt, using Equation 9.10:

$$VFA = 100 \frac{(VMA - VTM)}{VMA} = 100 \frac{(16.0 - 5.9)}{16.0} = 63.3$$

- A summary of all calculations is given in the following table:

P_b (%)	G_{mb}	Corrected Stability (kN)	Flow, (0.25 mm)	G_{mm}	G_{se}	VTM (%)	VMA (%)	VFA (%)
4.0	2.360	6.3	9	2.507		5.9	16.0	63.3
4.5	2.378	6.7	10	2.488		4.4	15.8	71.9
5.0	2.395	5.4	12	2.470	2.666	3.0	15.6	80.5
5.5	2.405	5.1	15	2.452		1.9	15.7	87.8
6.0	2.415	4.7	22	2.434		0.8	15.8	95.0

- Plot stability, flow and volumetric parameters versus P_b . (See Figure SP9.6.)
- Determine the asphalt content that corresponds to VTM = 4% and the corresponding parameters.
Compare with criteria:

	From Graphs	Criteria
P_b @ 4%	4.6	
Stability (kN)	6.6	5.34 (min)
Flow (0.25 mm)	10.5	8 to 16
G_{mb}	2.383	NA
VMA (%)	15.7	15.0 (min)
VFA (%)	75	65 to 78

Therefore, the design asphalt content is 4.6%.

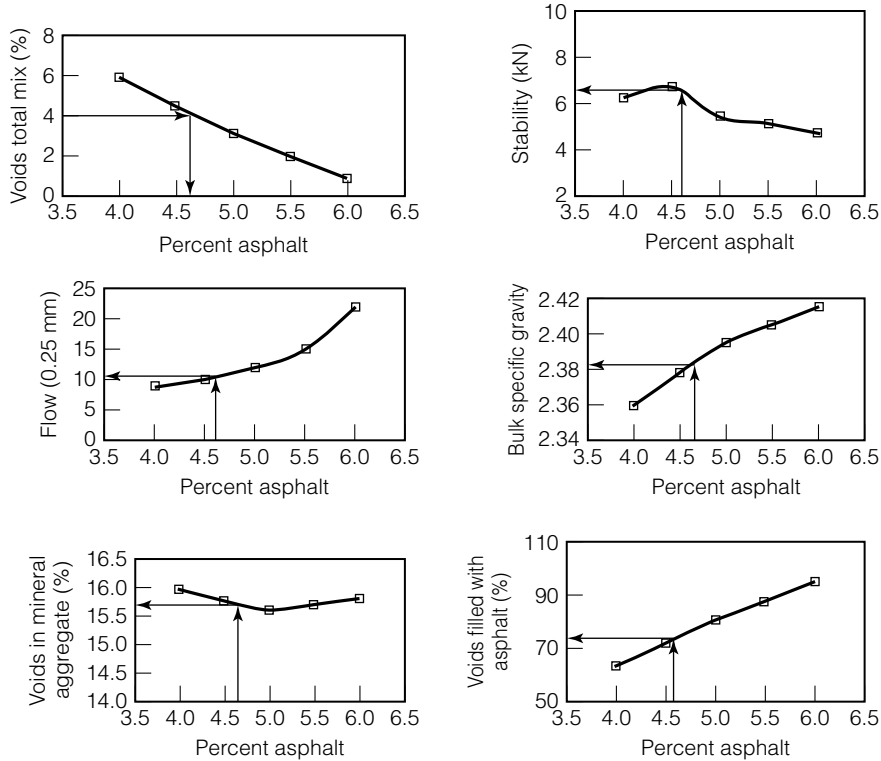


FIGURE SP9.6

9.9.6 Hveem Method of Mix Design

The basic steps required for performing Hveem mix design are the following (The Asphalt Institute 1995):

1. aggregate evaluation
2. asphalt cement evaluation
3. evaluation of centrifuge kerosene equivalent of fine aggregate
4. evaluation of surface capacity of coarse aggregate
5. estimation of optimum asphalt content
6. specimen preparation
7. measurement of the Hveem stability
8. density and voids analysis
9. determination of design asphalt content.

The evaluation of aggregate and asphalt cement is performed as in the Marshall method of mix design. The Hveem method requires measuring

aggregate properties and using a series of charts to estimate the design asphalt content (The Asphalt Institute 1995).

Three cylindrical specimens 102 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high are prepared, using the California kneading compactor (Figure 9.25) according to ASTM D1561. Three asphalt contents near the estimated design value are used to fabricate the specimens. The Hveem stability of the specimens is determined using the Hveem stabilometer (Figure 9.35), according to ASTM D1560. The Hveem stabilometer is a device that allows for the application of a lateral pressure on the specimen while applying vertical load using a compression machine.

As in the Marshall method, the bulk specific gravity, theoretical maximum specific gravity, percent air voids (VTM), and density of all specimens are determined. The Hveem stability, density, and air voids are tabulated and plotted versus asphalt content. The optimum asphalt content for the design mix should be the highest asphalt content the mix will accommodate without reducing the stability or void content below the minimum values required by the design criteria.

The laboratory-developed mixture design forms the basis for the initial JMF. The initial JMF should be adjusted to consider the slight differences between the laboratory-supplied aggregates and those used in the field.

9.9.7 ■ Evaluation of Moisture Susceptibility

Since loss of bond between asphalt and aggregates (*stripping*) has become a significant form of asphalt pavement distress, several methods have been developed for evaluating the susceptibility of a mix to water damage. Most of the popular methods require the specimens to be at the optimum asphalt content and mix gradation.



FIGURE 9.35 Hveem stabilometer.

The specimens are divided into two lots: reference specimens and conditioned specimens. A strength test is used to evaluate the strength before and after conditioning; the retained strength, the ratio of conditioned strength to reference strength, expressed in percent, is computed. Criteria are used to determine if the retained strength is adequate. The different techniques for evaluating moisture susceptibility vary, depending on the specimen preparation, conditioning procedures, and strength.

The immersion–compression test (ASTM D1075) has been used to evaluate moisture susceptibility. The method evaluates the retained compressive strength after vacuum saturation. Other methods use Marshall specimens, freezing and water soaking to condition the samples, and determining diametral strength and modulus values to evaluate the retained strength. Freezing the samples greatly increases the severity of the test.

There are several ways to alter asphalt concrete's susceptibility to water damage. Methods identified by the Asphalt Institute include the following:

1. increasing asphalt content
2. using a higher viscosity asphalt cement
3. cleaning aggregate of any dust and clay
4. adding antistripping additives
5. altering aggregate gradation

In addition, portland cement and lime have been used by some agencies as antistripping agents. Generally, when water damage susceptibility is a problem, the additive is added to the mix at three levels, and the water damage test is performed to determine the minimum amount of additive that can be used to increase the retained strength to an acceptable level. If an acceptable mix can be developed, Marshall or Hveem specimens are prepared, and the mix is tested to determine whether it meets the design criteria.

9.10 Characterization of Asphalt Concrete

Tests used to characterize asphalt concrete are somewhat different from those used to characterize other civil engineering materials, such as steel, portland cement concrete, and wood. One of the main reasons for this difference is that asphalt concrete is a nonlinear viscoelastic or viscoelastoplastic material. Thus, its response to loading is greatly affected by the rate of loading and temperature. Also, asphalt pavements are typically subjected to dynamic loads applied by traffic. Moreover, asphalt pavements do not normally fail due to sudden collapse under the effect of vehicular loads, but due to accumulation of permanent deformation in the wheel path (rutting), cracking due to repeated bending of the asphalt concrete layer (fatigue cracking), thermal cracking, excessive roughness of the pavement surface, migration of asphalt binder at the pavement surface (bleeding or flushing), loss of flexibility of asphalt binder due to aging and oxidation (raveling), loss of

bond between the asphalt binder and aggregate particles due to moisture (stripping), or other factors. Therefore, most of the tests used to characterize asphalt concrete try to simulate actual field conditions.

Many laboratory tests have been used to evaluate asphalt concrete properties and to predict its performance in the field. These tests are performed on either laboratory-prepared specimens or cores taken from in-service pavements. These tests measure the response of the material to load, deformation, or environmental conditions, such as temperature, moisture, or freeze and thaw cycles. Some of these tests are based on empirical relations, while others evaluate fundamental properties. All tests on asphalt concrete are performed at accurately controlled test temperatures and rates of loading, since asphalt response is largely affected by these two parameters.

The Superpave tests used for mix design, as well as Marshall or Hveem tests discussed earlier, have been used to characterize asphalt concrete mixtures. Other tests are also being used, some of which are standardized by ASTM or AASHTO, while others have been used mostly for research. The next several sections discuss some of the common tests.

9.10.1 ■ Indirect Tensile Strength

When traffic loads are applied on the pavement surface, tension is developed at the bottom of the asphalt concrete layer. Therefore, it is important to evaluate the tensile strength of asphalt concrete for the design of the layer thickness. In this test, a cylindrical specimen 102 mm (4 in.) in diameter and 64 mm (2.5 in.) high is typically used. A compressive vertical load is applied along the vertical diameter, using a loading device similar to that shown in Figure 9.36. The load is applied by using two curved loading strips moving with a rate of deformation of 51 mm/min. (2 in./min.). Tensile stresses are developed in the horizontal direction, and when these stresses reach the tensile strength, the specimen fails in tension along the vertical diameter. The test is performed at a specified temperature. With 12.5 mm (0.5 in.) loading strips, the indirect tensile strength is computed as

$$\sigma_t = \frac{2P}{\pi t D} \quad (9.24)$$

where

- σ_t = tensile strength, MPa (psi)
- P = load at failure, N (lb)
- t = thickness of specimen, mm (in.)
- D = diameter of specimen, mm (in.)

9.10.2 ■ Diametral Tensile Resilient Modulus

To evaluate the structural response of the asphalt pavement system, the modulus of asphalt concrete material is needed. Since asphalt concrete is not a linear viscoelastic material, the modulus of elasticity, Young's modulus, is not applicable. The diametral tensile resilient modulus test (ASTM

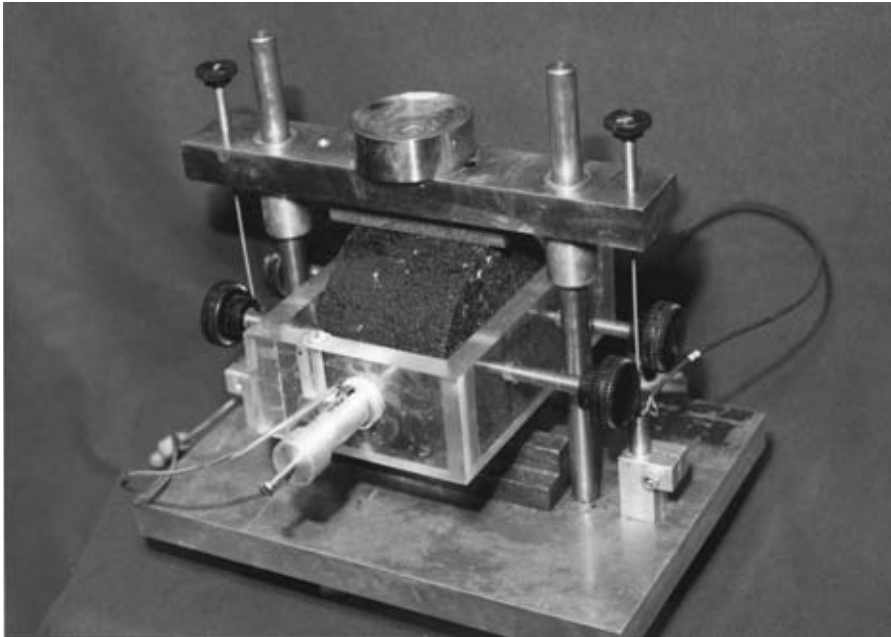


FIGURE 9.36 Diametral loading for indirect tensile strength and resilient modulus tests.

D4123) provides an analogous modulus, known as the resilient modulus. The test uses a cylindrical specimen 102 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high. A pulsating load is applied along the vertical diameter, using a load guide device similar to that shown in Figure 9.36. The load is commonly applied with a duration of 0.1 second and a rest period of 0.9 second. After a few hundred repetitions, the recoverable horizontal deformation is measured using two linear variable differential transducers (LVDTs). Figure 9.37 shows typical load and horizontal deformation versus time relationships. Since the test is nondestructive, the test is repeated on the same specimen after rotating it 90°. The test is commonly performed at three temperatures: 5°C, 25°C, and 40°C (41°F, 77°F, and 104°F). The diametral tensile resilient modulus is computed as

$$M_R = \frac{P(0.27 + \nu)}{t \cdot \Delta H} \quad (9.25)$$

where

M_R = indirect tensile resilient modulus, MPa (psi)

P = repeated load, N (lb)

ν = Poisson's ratio, typically 0.3, 0.35, and 0.4 at temperatures of 5°C, 25°C, and 40°C, respectively

t = thickness of specimen, mm (in.)

ΔH = sum of recoverable horizontal deformations on both sides of specimen, mm (in.)

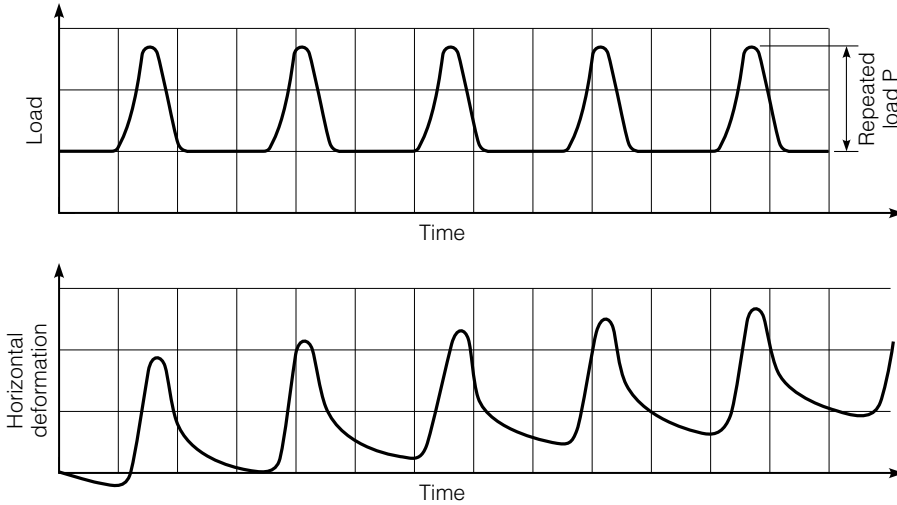


FIGURE 9.37 Typical load and horizontal deformation versus time during the resilient modulus test.

Typical resilient modulus values of asphalt concrete are 6.89 GPa, 4.13 GPa, and 1.38 GPa (1000 ksi, 600 ksi, and 200 ksi) at temperatures of 5°C, 25°C, and 40°C, respectively. The diametral tensile resilient modulus test is very sophisticated, because it measures very small deformations. Therefore, extreme caution must be exercised to align the specimen between the loading trips and to reduce possible rocking. Also, the load magnitude must be small enough to reduce the possibility of permanent deformation in the specimen, yet large enough to obtain measurable deformation.

Sample Problem 9.7

The resilient modulus test was performed on an asphalt concrete specimen and the following data were obtained:

- diameter = 4.000 in.
- thickness = 2.523 in.
- repeated load = 559 lb
- sum of recoverable horizontal deformations = 254×10^{-6} in.

Assuming a Poisson's ratio of 0.35, calculate the resilient modulus.

Solution

$$\begin{aligned}
 M_R &= \frac{P(0.27 + \nu)}{t \cdot \Delta H} = 559 \times (0.27 + 0.35) / (2.523 \times 254 \times 10^{-6}) \\
 &= 541,000 \text{ psi}
 \end{aligned}$$

9.10.3 ■ Freeze and Thaw Test

The freeze and thaw test is performed to evaluate the effect of freeze and thaw cycles on the stiffness properties of asphalt concrete. Cylindrical specimens 102 mm (4 in.) in diameter and 64 mm (2.5 in.) high are used. Three specimens are tested for resilient modulus as discussed earlier, while the other three specimens are subjected to cycles of freeze and thaw, after which the resilient modulus is determined. The tensile strength ratio is computed by dividing the average resilient modulus of conditioned specimens by the average resilient modulus of unconditioned specimens, expressed in percent. A minimum tensile strength ratio is usually required to identify mixes that are not severely affected by freeze and thaw cycles.

9.10.4 ■ Use of Rheological Models to Analyze Time-Dependent Response

Asphalt concrete is a viscoelastic material exhibiting a time-dependent response under load. Rheological models consisting of combinations of Hookean (spring) and Newtonian (dashpots) elements have been used to analyze the response of time-dependent materials, as discussed in Chapter 1. The Burgers model illustrated in Figure 1.12 can closely approximate the response of asphaltic mixtures (Mamlouk 1984). Laboratory tests, such as the creep test, are used to obtain the parameters of the Burgers model using a curve-fitting procedure. Once these parameters are determined, the model can be used to predict the response of the material under different loading conditions. For example, Burgers model has been used to predict rutting of asphalt concrete pavement under the action of traffic loads.

9.11 Asphalt Concrete Production

Asphalt concrete is produced in either a batch plant or a continuous (drum) plant (The Asphalt Institute 1989). In the United States, batch plants were used extensively in the past; however, more energy efficient continuous plants are now preferred.

In continuous plants (Figure 9.38), aggregates of different gradations are placed in cold bins. The gradation proportions needed are taken from the cold bins by a cold feed elevator. Aggregates are transferred to the first part of the drum, where they are dried and heated. Hot asphalt cement is introduced in the last one-third of the drum; then aggregates and asphalt are mixed. Since asphalt concrete is produced continuously in this type of plant, it is transferred to a storage silo until placed in a truck and transported to the job site.

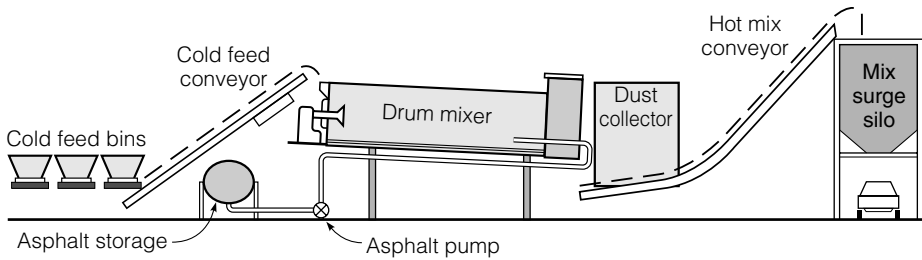


FIGURE 9.38 Continuous (drum) mix asphalt concrete plant.

9.12 Recycling of Asphalt Concrete

Recycling pavement materials has a long history. However, recycling became more important in the mid-1970s, after the oil embargo, due to the increase in asphalt prices. In an effort to efficiently use available resources, there was a need to recycle or reuse old pavement materials (The Asphalt Institute 1989). Although the pavement could be badly deteriorated, the old asphalt concrete materials could be successfully reused in new pavements. Currently, recycling of old pavement materials is becoming a normal practice due to the following advantages:

1. economic saving of about 25% of the price of materials
2. energy saving in manufacturing and transporting raw materials

3. environmental saving by reducing the amount of required new materials and by eliminating the problem of discarding old materials
4. eliminating the problem of reconstruction of utility structures, curbs, and gutters associated with overlays
5. reducing the dead load on bridges due to overlays
6. maintaining the tunnel clearance, compared with overlays

Recycling can be divided into three types: *surface recycling*, *central plant recycling*, and *in-place recycling*.

9.12.1 ■ Surface Recycling

Surface recycling is defined as the reworking of the top 25 mm (1 in.) of the pavement surface using a heater–scarifier. The heater planing machine heats the pavement surface, which repairs minor cracks and roughness. Usually, a rejuvenating agent is added after heating, followed by slight scratching of the surface and compaction.

9.12.2 ■ Central Plant Recycling

Central plant recycling is performed by milling the old pavement (Figure 9.39) and sending the reclaimed asphalt pavement (RAP) to a central asphalt concrete plant, where it is mixed with some form of rejuvenating agent or soft asphalt and aggregates to produce hot-mixed asphalt concrete. If the RAP materials are mixed with the aggregates in a conventional asphalt concrete plant, they will burn and produce smoke, causing significant



FIGURE 9.39 Milling old pavement.

environmental problems. Therefore, the RAP materials are added to the pugmill (mixer) in the batch plant or added at midlength in the drum at the drum mix plant. The amount of recycled materials varies from 20% to 70%. The gradation of new aggregates is selected to correct any deficiency in the gradation of recycled aggregate. Typically, the grade of new asphalt cement is soft so that, when it is mixed with the old, hard asphalt, an appropriate consistency will result. The mix design of asphalt concrete, including recycled materials, is usually performed using either the Marshall or Hveem procedure.

In addition to hot-mix central plant recycling, cold central plant recycling can use new emulsified or cutback asphalt. However, the cold process will not have the quality of the hot-mixed material.

9.12.3 ■ In-Place Recycling

In-place recycling is performed by ripping and pulverizing the old pavement surface and adding new aggregate, water, and asphalt emulsion. The old and new materials are mixed together in place, graded, and compacted. The surface is left to cure and is then used as a surface layer for low-volume roads. The recycled layer can also be used as a stabilized base, covered by an asphalt concrete surface.

9.13 Additives

Many types of additives (modifiers) are used to improve the properties of asphalt or to add special properties to the asphalt concrete mixtures (Roberts et al. 1996). Laboratory tests are usually performed and field performance is observed in order to evaluate the effect of the additives and to justify their cost. The effects of using additives should be carefully evaluated; otherwise premature pavement failure might result. The recyclability of modified asphalt mixtures is still being evaluated.

9.13.1 ■ Fillers

Several types of fillers, such as crushed fines, portland cement, lime, fly ash, and carbon black can be added to asphalt concrete. Fillers are used to satisfy gradation requirements of materials passing the 0.075 mm (No. 200) sieve, to increase stability, to improve bond between aggregates and asphalt, or to fill the voids and thus reduce the required asphalt.

9.13.2 ■ Extenders

Extenders such as sulfur and lignin are used to reduce the asphalt requirements, thus reducing the cost.

9.13.3 ■ Rubber

Rubber has been used in asphalt concrete mixture in the form of natural rubber, styrene–butadiene (SBR), styrene–butadiene–styrene (SBS), or recycled tire rubber. Rubber increases elasticity and stiffness of the mix and increases the bond between asphalt and aggregates. Scrap rubber tires can be added to the asphalt cement (wet method) or added as crumb rubber to the aggregates (dry method).

9.13.4 ■ Plastics

Plastics have been used to improve certain properties of asphalt. Plastics used include polyethylene, polypropylene, ethyl–vinyl–acetate (EVA), and polyvinyl chloride (PVC). They increase the stiffness of the mix, thus reducing the rutting potential. Plastics also may reduce the temperature susceptibility of asphalt and improve its performance at low temperatures.

9.13.5 ■ Antistripping Agents

Antistripping agents are used to improve the bond between asphalt cement and aggregates, especially for water susceptible mixtures. Lime is the most commonly used antistripping agent and can be added as a filler or a lime slurry and mixed with the aggregates. Portland cement can be used as an alternative to lime.

9.13.6 ■ Others

Other additives, such as fibers, oxidants, antioxidants, and hydrocarbons, have been used to modify certain asphalt properties' tensile strength and stiffness.

S U M M A R Y

Asphalt produced from crude oil is a primary road-building material. The civil engineer is directly involved with the specification and requirements for both the asphalt cement binder and the asphalt concrete mixtures. There are several methods for grading asphalt cements. The current trend is toward the use of the performance-grading method, used in the Superpave process developed through the Strategic Highway Research Program. This grading method directly ties the binder properties to pavement-performance parameters. Similarly, the Superpave mix design method uses performance

tests to evaluate the mixture characteristics relative to expected field performance. This method will continue to be refined by highway agencies. With the support of the Federal Highway Administration, there is a concerted effort being placed on replacing the traditional Marshall and Hveem mix design methods with the Superpave procedures.

Q U E S T I O N S A N D P R O B L E M S

- 9.1 What is the difference between tar and asphalt cement?
- 9.2 Discuss the main uses of asphalt.
- 9.3 Define what is meant by temperature susceptibility of asphalt. Discuss the effect on the performance of asphalt concrete pavements. Are soft asphalts used in hot or cold climates?
- 9.4 Temperature has a large effect on the asphalt viscosity. On one graph, plot the relationship between asphalt viscosity (logarithmic) and temperature for two cases: (a) a low-temperature susceptible asphalt and (b) a high-temperature susceptible asphalt. Label all axes and relations.
- 9.5 Briefly discuss the chemical composition of asphalt.
- 9.6 What is the significance of each one of these tests:
 - a. flash point test
 - b. RTFO procedure
 - c. rotational viscometer test
 - d. dynamic shear rheometer test
 - e. penetration test.
- 9.7 Discuss the aging that occurs in asphalt cement during mixing with aggregates and in service. How can the different types of aging of asphalt cement be simulated in the laboratory?
- 9.8 Show how various Superpave tests used to characterize the asphalt binder are related to pavement performance.
- 9.9 Define the four methods used to grade asphalt binders. Which method is used in your state?
- 9.10 To select an asphalt binder for a specific location, the mean seven-day maximum pavement temperature is estimated at 61°C with a standard deviation of 1.5°C . The mean minimum pavement temperature is -8°C with standard deviation of 3.2°C . What grade asphalt (PG) is needed at 98% reliability.

- 9.11 As a materials engineer working for a highway department, what standard PG asphalt binder grade would you specify for each of the conditions shown in Table P9.11 (show all calculations and fill in the table)?

Table P9.11

Case	Seven-Day Pavement		Maximum Temperature, °C		Minimum Pavement Temperature, °C		Recommended PG Grade	
	Mean, °C	Std. Dev., °C	Mean, °C	Std. Dev., °C	50% Reliability	98% Reliability		
1	46	1.5	-34	2				
2	56	2	-15	2.5				
3	66	2	8	1.5				

- 9.12 For the following temperature conditions, calculate the proper Super-pave grade for both 50% and 98% reliabilities (show your calculation).
- Seven-day maximum pavement temperature of 46°C with a standard deviation of 2°C.
 - Minimum pavement temperature of 19°C with a standard deviation of 4°C.
- 9.13 What are the differences between CRS-2 and SS-1 emulsions?
- 9.14 Discuss how asphalt emulsions work as a binder in asphalt mixtures.
- 9.15 What are the components of hot-mix asphalt? What is the function of each component in the mix?
- 9.16 What are the objectives of the asphalt concrete mix-design process?
- 9.17 Why is it important to have an optimum binder content in HMA? What would happen if a less-than-optimum binder content is used? What would happen if more than the optimum value is used? What is the typical range of binder content in HMA?
- 9.18 Explain why the strength of asphalt concrete is not necessarily the most important property of the material.
- 9.19 An asphalt concrete specimen has a mass in air of 1249.3 g, mass in water of 735.8 g, and saturated surface-dry mass of 1250.2 g. Calculate the bulk specific gravity of the specimen.
- 9.20 As part of mix design, a laboratory-compacted cylindrical asphalt specimen is weighed for determination of bulk specific gravity. The following numbers are obtained:
- Dry Mass in Air = 1204.5 grams
 - Mass when submerged in water = 689.4 grams
 - Mass of Saturated Surface Dry (SSD) = 1211.3 grams
- a. What is the bulk specific gravity of the compacted specimen (G_{mb})
 - b. If the maximum theoretical specific gravity of the specimen (G_{mm}) is 2.531, what would be the air void content of the specimen in percent?

- 9.21 For asphalt concrete, define
- air voids
 - voids in the mineral aggregate
 - voids filled with asphalt
- 9.22 An aggregate blend is composed of 53% coarse aggregate by weight (Sp. Gr. 2.702), 43% fine aggregate (Sp. Gr. 2.621), and 4% filler (Sp. Gr. 2.779). The compacted specimen contains 6% asphalt binder (Sp. Gr. 1.052) by weight of total mix, and has a bulk density of 145.2 lb/ft³. Ignoring absorption, compute the percent voids in total mix, percent voids in mineral aggregate, and the percent voids filled with asphalt.
- 9.23 An asphalt concrete mixture includes 94% aggregate by weight. The specific gravities of aggregate and asphalt are 2.7 and 1.0, respectively. If the bulk density of the mix is 145 pcf, what is the percent voids in the total mix?
- 9.24 After two years of traffic, cores were recovered from the roadway which has severe rutting and bleeding. The bulk specific gravity and the maximum theoretical specific gravity were measured on these cores and are as follows:
- Bulk specific gravity = 2.498
Maximum theoretical specific gravity = 2.545
- Calculate the air voids
 - If the design air void content was 4%, explain what effect the calculated air voids had on the rutting and bleeding noted.
- 9.25 An asphalt concrete specimen has the following properties:
- Asphalt content = 5.3% by total weight of mix
Bulk specific gravity of the mix = 2.442
Theoretical maximum specific gravity = 2.535
Bulk specific gravity of aggregate = 2.703
- Calculate the percents VTM, VMA, and VFA.
- 9.26 Briefly describe the volumetric mix design procedure of Superpave.
- 9.27 Based on the data shown in Table P9.27, select the blend for a Superpave design aggregate structure.

Table P9.27

Data	Blend		
	1	2	3
G_{mb}	2.451	2.465	2.467
G_{mm}	2.585	2.654	2.584
G_b	1.030	1.030	1.030
P_b	5.9	5.5	5.8
P_s	94.1	94.5	94.2
P_d	4.5	4.5	4.5
G_{sb}	2.657	2.667	2.705
H_{ini}	127	135	124
H_{des}	113	114	118

9.28 Based on the data in Table P9.28, determine the design binder content for a Superpave mix design for a 15 million ESAL.

Table P9.28

Data	Binder content (%)			
	5.5	6.0	6.5	7.0
G_{mb}	2.351	2.441	2.455	2.469
G_{mm}	2.579	2.558	2.538	2.518
G_b	1.025	1.025	1.025	1.025
P_s	94.5	94.0	93.5	93.0
P_d	4.5	4.5	4.5	4.5
G_{sb}	2.705	2.705	2.705	2.705
h_{ini}	129	131	131	128
h_{des}	112	113	116	115

9.29 Given the data in Table P9.29, select the blend and the design binder content for a Superpave design aggregate structure for a 5 million ESAL.

Table P9.29

Data	Blend		
	1	2	3
G_{mb}	2.457	2.441	2.477
G_{mm}	2.598	2.558	2.664
G_b	1.025	1.025	1.025
P_b	5.9	5.7	6.2
P_s	94.1	94.3	93.8
P_d	4.5	4.5	4.5
G_{sb}	2.692	2.688	2.665
H_{ini}	125	131	125
H_{des}	115	118	115

Data	Binder Content (%)			
	5.4	5.9	6.4	6.9
G_{mb}	2.351	2.441	2.455	2.469
G_{mm}	2.570	2.558	2.530	2.510
G_b	1.025	1.025	1.025	1.025
P_s	94.6	94.1	93.6	93.1
P_d	4.5	4.5	4.5	4.5
G_{sb}	2.688	2.688	2.688	2.688
h_{ini}	125	131	126	130
h_{des}	115	118	114	112

- 9.30 The Marshall method of mix design has been widely used by many highway agencies.
- What are the steps of Marshall Mix Design?
 - What parameters are calculated?
 - Show the typical graphs that are plotted after tests are completed.
 - What is the purpose of plotting these graphs?
- 9.31 An asphalt concrete mixture is to be designed according to the Marshall procedure. An AC-20 asphalt cement with a specific gravity of 1.00 is to be used. A dense aggregate blend is to be used, with a maximum size of 19 mm and bulk specific gravity of 2.696. The theoretical maximum specific gravity of the mix is 2.470. Trial mixes were made, with the average results shown in Table P9.31. Using a spreadsheet program, plot the appropriate graphs necessary for the Marshall procedure, and select the optimum asphalt content using the Asphalt Institute design criteria for medium traffic. Assume a design air void content of 4% when using Table 9.14.

Table P9.31

Asphalt Content, % by Weight	Bulk Specific Gravity	Stability, N	Flow, 0.25 mm
4.0	2.303	7076	9
4.5	2.386	8411	10
5.0	2.412	7565	12
5.5	2.419	5963	15
6.0	2.421	4183	22

- 9.32 An asphalt concrete mixture is to be designed according to the Marshall procedure. An AC-30 asphalt cement with a specific gravity (G_b) of 1.00 is to be used. A dense aggregate blend is to be used, with a maximum aggregate size of 3/4 in. and a bulk specific gravity (G_{sb}) of 2.786. The theoretical maximum specific gravity of the mix (G_{mm}), at asphalt content of 5.0%, is 2.490. Trial mixes were made with average results as shown in Table P9.32. Using a spreadsheet program, plot the appropriate six graphs necessary for the Marshall procedure and select the optimum asphalt content, using the Asphalt Institute design criteria for medium traffic (see Table 9.13). Assume a design air void content of 4% when using Table 9.14.

Table P9.32

Asphalt Content (P_b) (% by Weight)	Bulk Specific Gravity (G_{mb})	Stability, lb	Flow, 0.01 in.
3.50	2.294	1600	8
4.00	2.396	1980	9
4.50	2.421	2130	11
5.00	2.416	1600	14
5.50	2.401	1280	20

- 9.33 The Marshall procedure was used to design an asphalt concrete mixture for a heavy-traffic road. Asphalt cement with a specific gravity of 1.025 is to be used. The mixture contains a 19 mm nominal maximum particle size aggregate, with bulk specific gravity of 2.654. The theoretical maximum specific gravity of the mix is 2.480 at 4.5% asphalt content. Trial mixes were made, with the average results shown in Table P9.33. Determine the optimum asphalt content using the Asphalt Institute design criteria. Assume a design air void content of 4% when using Table 9.14.

Table P9.33

Asphalt Content, % by Weight	Bulk Specific Gravity	Stability, kN	Flow, 0.25 mm
3.5	2.367	8.2	7.3
4.0	2.371	8.6	9.4
4.5	2.389	7.5	11.5
5.0	2.410	7.2	12.5
5.5	2.422	6.9	13.2

- 9.34 Describe the process for determining the stripping potential for an asphalt concrete mix.
- 9.35 Briefly discuss how the indirect tensile resilient modulus is determined in the lab.
- 9.36 The resilient modulus test was performed on an asphalt concrete specimen and the following data were obtained:
- Diameter = 4.029 in.
 Height = 2.497 in.
 Repeated load = 559 lb
 Sum of recoverable horizontal deformations = 254×10^{-6} in.
- Assuming a Poisson's ratio of 0.35, calculate the resilient modulus.
- 9.37 State six advantages of recycling asphalt pavement materials. Why can we not mix the RAP materials with aggregates in a conventional hot-mix asphalt concrete plant? Show the proper ways of recycling the RAP materials in the two types of hot-mix asphalt plants.
- 9.38 State four different asphalt modifiers that can be added to asphalt or asphalt mixtures, and indicate the effect of each.
- 9.39 When is portland cement used in asphalt concrete?
- 9.40 During construction of the HMA layer of asphalt pavement, cores were taken at random after compaction to detect any construction problem and to ensure that the density is within the specification limits. The target value was set at 148.2 pcf and the upper and lower specification limits were set at 148.9 pcf and 147.5 pcf, respectively. The density data shown in Table P9.40 were collected.

Table P9.40

Core No.	Density (pcf)	Core No.	Density (pcf)
1	148.3	11	147.8
2	147.8	12	148.1
3	148.2	13	147.3
4	148.7	14	147.7
5	148.2	15	147.3
6	147.7	16	147.0
7	148.4	17	147.1
8	147.8	18	146.7
9	147.7	19	146.9
10	148.6	20	146.8

- a. Using a spreadsheet program, create a control chart for these data showing the target value and the upper and lower specification limits. Are the density data within the specification limits?
- b. Comment on any trend and possible reasons.

9.14 References

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10

WOOD

Wood, because of its availability, relatively low cost, ease of use, and durability, if properly maintained, continues to be an important civil engineering material. Wood is used extensively for buildings, bridges, utility poles, floors, roofs, trusses, and piles. (See Figures 10.1 and 10.2.) Civil engineering applications include both natural wood and engineered wood products, such as laminates, plywood, and strand board. In order to use wood efficiently, it is important to understand its basic properties and limitations. In the United States, the Forest Service of the Department of Agriculture has broad management responsibility for the harvesting of wood from public lands and for assisting private sources with the selection of products for harvesting. This agency has produced an excellent document describing the characteristics and properties of wood (USDA-FS, 1999).

This chapter covers the properties and characteristics of wood. In the design of a wood structure, joints and connections often limit the design elements. These are generally covered in a design class for wood construction and, therefore, are not considered in this text.

Wood is a natural, renewable product from trees. Biologically, a tree is a woody plant that attains a height of at least 6 m (20 ft), normally has a single self-supporting trunk with no branches for about 1.5 m (4 ft) above the ground, and has a definite crown. There are over 600 species of trees in the United States.

Trees are classified as either endogenous or exogenous, based on the type of growth. Endogenous trees, such as bamboo, grow with intertwined fibers. Wood from endogenous trees is not generally used for engineering applications in the United States. Exogenous trees grow from the center out by adding concentric layers of wood around the central core. This book considers only exogenous trees.

Exogenous trees are broadly classified as deciduous and conifers, producing hardwoods and softwoods, respectively. The terms *hardwood* and *softwood* are classifications within the tree family, not a description of the woods' characteristics. In general, softwoods are softer, less dense, and easier



FIGURE 10.1 Wood frame used for building structural support.



FIGURE 10.2 Wooden roller coaster.

to cut than hardwoods. However, exceptions exist such as Balsa, a very soft and lightweight wood that is technically a hardwood.

Deciduous trees generally shed their leaves at the end of each growing season. Commercial hardwood production in the U.S. comes from 40 different tree species. Hardwoods are generally used for furniture and decorative veneers, due to their pleasing grain pattern. The cost of hardwoods limits their construction application.

Conifers, also known as evergreens, have needlelike leaves and normally do not shed them at the end of the growing season. Conifers grow continuously through the crown, producing a uniform stem and homogeneous characteristics (Panshin & De Zeeuw, 1980). Softwood production in the U.S. comes from about 20 individual species of conifers. Conifers are widely used for construction. Conifers grow in large stands, permitting economical harvesting. They mature rapidly, making them a renewable resource. Table 10.1 shows examples of hardwood and softwood species (USDA-FS, 1999).

10.1 Structure of Wood

Wood has a distinguished structure that affects its use as a construction material. Civil and construction engineers need to understand the way the tree grows and the anisotropic nature of wood in order to properly design and construct wood structures.

10.1.1 Growth Rings

The concentric layers in the stem of exogenous trees are called *tree rings* or *annual rings*, as shown in Figure 10.3a. The wood produced in one growing season constitutes a single growth ring. Each annual ring is composed of early wood, produced by rapid growth during the spring, and latewood from summer growth. Latewood consists of dense, dark, and thick-walled cells producing a stronger structure than early wood, as shown in Figure 10.3b. The predominant physical features of the tree stem include the *bark*, *cambium*, *wood*, and *pith*, as shown in Figure 10.4. The bark is the exterior covering of the tree and has an outer and an inner layer. The outer layer is dead and corky and has great variability in thickness, dependent on the species and age of the tree. The inner bark layer is the growth layer for bark but is not part of the wood section of the tree. The cambium is a thin layer of cells situated between the wood and the bark and is the location of all wood growth.

The wood section of the tree is composed of *sapwood* and *heartwood*. Sapwood functions as a storehouse for starches and as a pipeline to transport sap. Generally, faster growing species have thick sapwood regions. In

TABLE 10.1 Major Sources of Hardwood and Softwood Species by Region

		Softwoods			
		Hardwoods			
Western	Northern and Appalachia	Southern	Western	Northern	Southern
Ash	Ash	Red alder	Incense cedar	Northern white cedar	Atlantic white cedar
Basswood	Aspen	Oregon ash	Port Orford cedar	Balsam fir	Bald cypress
American beech	Basswood	Aspen	Douglas-fir	Eastern hemlock	Fraser fir
Butternut	Buckeye	Black cottonwood	Western hemlock	Fraser fir	Southern pine
Cottonwood	Butternut	California black oak	Western larch	Jack pine	Eastern red cedar
Elm	American beech	Big leaf maple	Lodgepole pine	Eastern white pine	
Hackberry	Birch	Paper birch	Ponderosa pine	Eastern red cedar	
Pecan hickory	Black cherry	Tan oak	Sugar pine	Eastern spruces	
True hickory	American chestnut*		Western white pine	Tamarack	
Honey locust	Cottonwood		Western red cedar		
Black locust	Elm		Red wood		
Magnolia	Hackberry		Englemann spruce		
Soft maple	True hickory		Siitka spruce		
Red oak	Honey locust		Yellow cedar		
White oak	Black locust				
Sassafras	Hard maple				
Sweetgum	Soft maple				
American sycamore	Red oak				
Tupelo	White oak				
Black walnut	American sycamore				
Black willow	Black walnut				
Yellow poplar	Yellow poplar				

* Chestnut no longer harvested, but lumber from salvage timbers available.

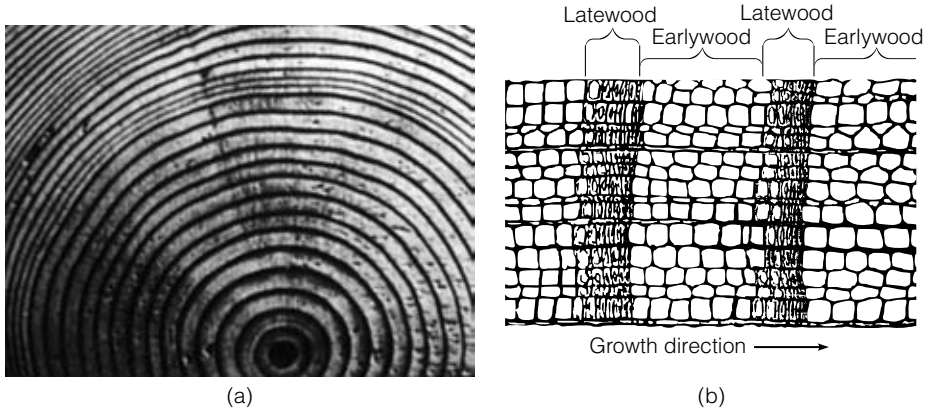


FIGURE 10.3 Cross section of a typical tree stem: (a) annual rings (photo courtesy of American Forest & Paper Association, Washington, D.C.) and (b) earlywood and latewood.

its natural state, sapwood is not durable when exposed to conditions that promote decay. Heartwood is not a living part of the tree. It is composed of cells that have been physically and chemically altered by mineral deposits. The heartwood provides structural strength for the tree. Since the heartwood does not contain sap, it is naturally resistant to decay.

The pith is the central core of the tree. Its size varies with the tree species, ranging from barely distinguishable to large and conspicuous. The color ranges from blacks to whitish, depending on the tree species and locality. The pith structure can be solid, porous, chambered, or hollow.

10.1.2 ■ Anisotropic Nature of Wood

Wood is an anisotropic material in that it has different and unique properties in each direction. The three axis orientations in wood are longitudinal or parallel to the grain, radial or cross the growth rings, and tangential or tangent to the growth rings, as illustrated in Figure 10.4. The anisotropic nature of wood affects physical and mechanical properties such as shrinkage, stiffness, and strength.

The anisotropic behavior of wood is the result of the tubular geometry of the wood cells. The wood cells have a rectangular cross section. The centers of the tubes are hollow, whereas the ends of the tubes are tapered. The length-to-width ratio can be as large as 100. The long dimension of the majority of cells is parallel to the tree's trunk. However, a few cells, in localized bundles, grow radially, from the center to the outside of the trunk. The preponderance of cell orientation in one direction gives wood its anisotropic characteristics. The hollow tube structure is very efficient in resisting compressive stresses parallel to its length, but readily deforms when loaded on its side. Also, fluctuations in moisture contents flex the tube walls but have little effect on the length of the tube.

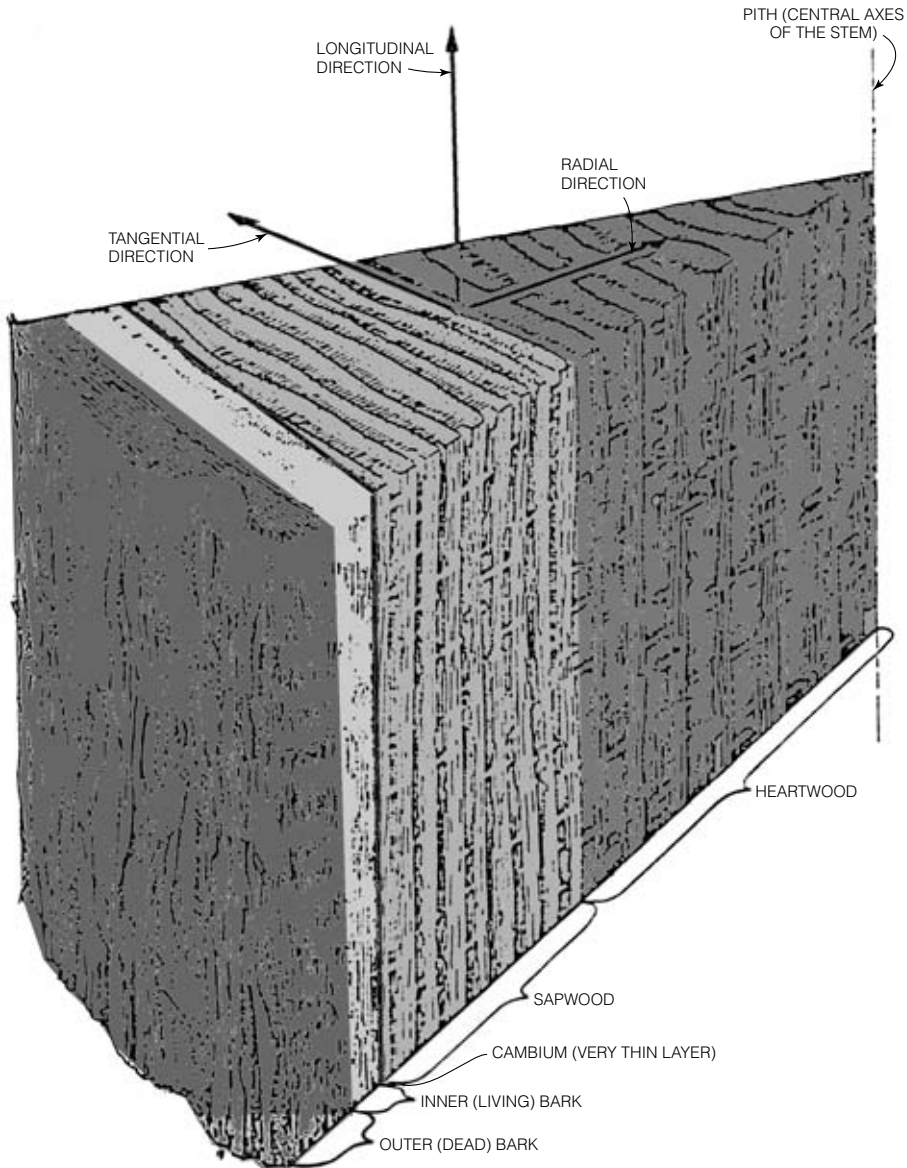


FIGURE 10.4 Main structural features of a typical tree stem.

10.2 Chemical Composition

Wood is composed of cellulose, lignin, hemicellulose, extractives, and ash producing minerals. Cellulose accounts for approximately 50 percent of the wood substance by weight (USDA-FS, 1999). The exact percent is species

dependent. It is a linear polymer (aliphatic carbon compound) having a high molecular weight. The main building block of cellulose is sugar–glucose. As the tree grows, linear cellulose molecules arrange themselves into highly ordered strands, called *fibrils*. These ordered strands form the large structural elements that compose the cell walls of wood fibers.

Lignin accounts for 23% to 33% of softwood and 16% – 25% of hardwood by weight. Lignin is mostly an intercellular material. Chemically, lignin is an intractable, insoluble, material that is loosely bonded to the cellulose. Lignin is basically the glue that holds the tubular cells together. The longitudinal shear strength of wood is limited by the strength of the lignin bounds.

Hemicelluloses are polymeric units made from sugar molecules. Hemicellulose is different from cellulose in that it has several sugars tied up in its cellular structure. Hardwood contains 20% to 30% hemicellulose and softwood averages 15% to 20%. The main sugar units in hardwood and softwood are xylose and monnose, respectively.

The extractives compose 5% to 30% of the wood substance. Included in this group are tannins and other polyphenolics, coloring matters, essential oils, fats, resins, waxes, gums, starches, and simple metabolic intermediates. These materials can be removed with simple inert neutral solvents, such as water, alcohol, acetone, and benzene. The amount contained in an individual tree depends on the species, growth conditions, and time of year the tree is harvested.

The ash-forming materials account for 0.1% – 3.0% of the wood material and include calcium, potassium, phosphate, and silica.

10.3 Moisture Content

The moisture content of a wood specimen is the weight of water in the specimen expressed as a percentage of the oven-dry weight of the wood. An oven-dried wood sample is a sample that has been dried in an oven at 100°C to 105°C (212°F to 220°F) until the wood attains a constant weight. Physical properties such as weight, shrinkage, and strength depend on the moisture content of wood.

Moisture exists in wood as either *bound* or *free water*. Bound water is held within the cell wall by adsorption forces, whereas free water exists as either condensed water or water vapor in the cell cavities. In green wood, the cell walls are saturated. However, the cell cavities may or may not contain free water. The level of saturation at which the cell walls are completely saturated, but no free water exists in the cell cavities, is called the *fiber saturation point* (FSP). FSP varies from species to species, but is typically in the range of 21% to 32%. The FSP is of great practical significance, because the addition or removal of moisture below the FSP has a large effect on practically all physical and mechanical properties of wood, whereas above the FSP, the properties are independent of moisture content.

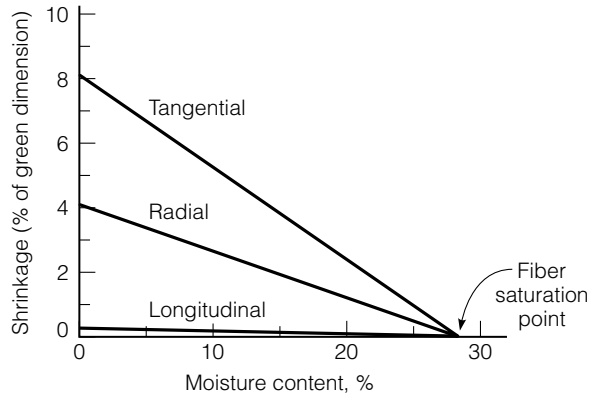


FIGURE 10.5 Relation between shrinkage and moisture content.

When the moisture content of wood is above the fiber saturation point, the wood is dimensionally stable. However, moisture fluctuations below the FSP always result in dimensional changes. Shrinkage is caused by loss of moisture from the cell walls, and conversely, swelling is caused by the gain of moisture in the cell walls. Figure 10.5 shows that the changes in wood dimensions vary from one direction to another. Dimensional changes in the radial direction are generally one-half the change in the tangential direction. Swelling and shrinkage in the longitudinal direction is minimal, typically 0.1% to 0.2% for a change in the moisture content from the FSP to oven dry. This anisotropy of dimensional changes of wood causes warping, checking, splitting, and structural performance problems, as discussed in more detail later in the chapter. It is also the reason that the sawing pattern of boards affects the amount of distortion when subjected to changes in moisture.

The moisture content in wood varies depending on air temperature and humidity. However, the natural change of moisture content is a slow process, so as atmospheric conditions change, the moisture content in wood tends to adjust to conditions near the average. The moisture content for the average atmospheric conditions is the *equilibrium moisture content (EMC)*. The Forest Service has developed values of the EMC as a function of temperature and humidity. The EMC ranges from less than 1%, at temperatures greater than 55°C (130°F) and 5% humidity, to over 20% at temperatures less than 27°C (80°F) and 90% humidity.

Sample Problem 10.1

A 250-mm wide Red Spruce plank is cut in such a way that the width is in the tangential direction of annual rings. Compute the change in width as the moisture content changes from 15% to 32%. (The fiber saturation point of Red Spruce is 27% and it shrinks 7.8% in drying from FSP to oven dry in the tangential direction.)

Solution

Swell does not occur above FSP. Increasing the moisture content from 15% to 27% causes swell. From the data given, Red Spruce swells 7.8% for a 27% change in moisture content (from zero moisture content to FSP).

$$\text{Percent swell} = (7.8/27) \times (27 - 15) = 3.5\%$$

$$\text{Change in dimension} = 250 \times (3.5/100) = 8.7 \text{ mm}$$

$$\text{New dimension} = 258.7 \text{ mm}$$

10.4 Wood Production

Trees are harvested in the fall or winter, because of their water content and environmental concerns connected with fire hazard and other plant growth. A vast industry has developed to harvest and process wood. Wood is harvested from forests as *logs*. They are transported to saw mills, where they are cut into dimensional shapes to produce a variety of products for engineering applications:

1. *Dimensional lumber* is wood from 50 mm to 125 mm (2 in. to 5 in.) thick, sawn on all four sides. Common shapes include 2×4 , 2×6 , 2×8 , 2×10 , 2×12 , and 4×4 .¹ These sizes refer to the rough-sawn dimensions of the lumber in inches. The rough-sawn lumber is surfaced to produce smooth surfaces; this removes 5 mm to 10 mm (1/4 in. to 3/8 in.) per side. For example, the actual dimensions of a 2×4 are 40 mm by 90 mm ($1\frac{1}{2}$ in. by $3\frac{1}{2}$ in.). Dimensional lumber is produced in lengths of 2.4 m to 7.2 m (8 ft. to 24 ft.) in 0.6 m (2 ft.) increments. Dimensional lumber is typically used for studs, sill and top plates, joints, beams, rafters, trusses, and decking.
 2. *Heavy timber* is wood sawn on all four sides; common shapes include 4×6 , 6×6 , 8×8 , and larger. As with the case of dimensional lumber, these sizes specify rough-sawn dimensions in inches. Surfacing generally removes 10 mm (3/8 in.) per side. Heavy timbers are used for heavy frame construction, landscaping, railroad ties, and marine construction.
 3. *Round stock* consists of posts and poles used for building poles, marine piling, and utility poles.
 4. *Engineered wood* consists of products manufactured by bonding together wood strands, veneers, lumber, and other forms of wood fiber to produce a larger and integral composite unit. These products are engineered and
1. The standards for dimensional lumber and heavy timber standards were implemented in about 1970. When remodeling older structures, the dimension of the existing lumber must be measured.

tested to have specific mechanical responses to loads. Structural engineered wood products include the following:

- structural panels including plywood, oriented strand board, and composite panels,
 - glued laminated timber (glulam),
 - structural composite lumber, and
 - composite structural members.
5. Specialty items are milled and fabricated products to reduce on-site construction time, includes lattice, handrails, spindles, radius edge decking, turned posts, etc.

Sawn wood production includes the following steps:

- Sawing into desired shape
- Seasoning
- Surfacing
- Grading
- Preservative treatment (optional)

Surfacing (planing) of the wood surface, to produce a smooth face, can be done before or after drying. Post drying surfacing is superior, because it removes small defects developed during the drying process. When surfacing is done before seasoning, the dimensions are slightly increased to compensate for shrinkage during seasoning.

10.4.1 ■ Cutting Techniques

The harvested wood is cut into lumber and timber at saw mills using circular saws, band saws, or frame saws. The most common patterns for sawing a log are *plain (slash)*, *quarter*, and *combination sawing*, as shown on Figure 10.6. (Levin, 1972).

The quality of the boards is related to the angle the growth rings make with the face of the board (i.e., the angle between the growth ring and the saw blade). There are three categories, as illustrated in Figure 10.7:

1. Flat-sawn, 45° or less
2. Rift-sawn, 45° to 80°
3. Vertical- or edge-sawn, 80° to 90°

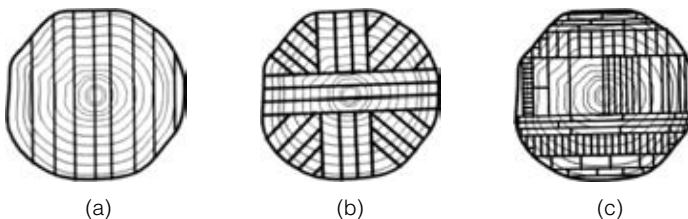


FIGURE 10.6 Common log sawing patterns: (a) plain sawing, (b) quarter sawing, and (c) combination sawing.

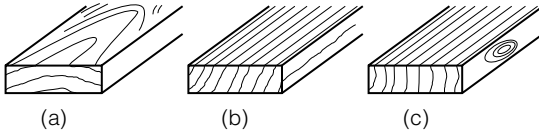


FIGURE 10.7 Types of board cut: (a) flat sawn, (b) rift sawn, and (c) vertical- or edge-sawn.

Flat-sawn boards have desirable exposure of grain for decorative applications. However, flat sawn boards tend to distort more than vertical-sawn boards in response to moisture fluctuations. Hence, vertical-sawn boards are generally better for structural applications.

The sawing pattern selected depends on the cross section of the tree, the capability of the mill, and the desired product. Plain sawing is rapid and economic, whereas quarter sawing maximizes the amount of vertical-sawn cuts. Some of the advantages of the different sawing patterns are summarized in Table 10.2 (USDA-FS, 1999).

10.4.2 ■ Seasoning

Green wood, in living trees, contains from 30% to 200% moisture by the oven-dry weight. Seasoning removes the excess moisture from wood. For structural wood, the recommended moisture content varies from 7% in the

TABLE 10.2 Relative Advantages of Plain-sawn and Quarter-sawn Lumber

Plain-sawn	Quarter-sawn
Shrinks and swells less in thickness	Shrinks and swells less in width
Surface appearance less affected by round or oval knots, compared to effect of spike knots in quartersawn boards; boards with round or oval knots not as weak as boards with spike knots	Cups, surface checks, and splits less in seasoning and in use
Shakes and pitch pockets, when present, extend through fewer boards	Raised grain caused by separation in annual rings does not become as pronounced
Figure patterns resulting from annual rings and some other types of figure brought out more conspicuously	Figure patterns resulting from pronounced rays, interlocked grain, and wavy grain are brought out more conspicuously
Less susceptible to collapse in drying	Does not allow liquids to pass through readily in some species
Costs less because it is easy to obtain	Holds paint better in some species
	Sapwood appears in boards at edges and its width is limited by the width of the log

dry southwestern states to 14% in the damp coastal regions. However, as it leaves the mill, framing lumber typically has an average moisture content of 15%.

Wood is seasoned by air and kiln drying. Air drying is inexpensive, but slow. The green lumber is stacked in covered piles to dry. These piles of lumber are made of successive layers of boards separated by 25 mm (1 inch) strips so that air can flow between the layers. The time required for drying varies with the climate and temperature of the area. Normally, three to four months is the maximum air drying time used in the U.S. Air drying is complete when the moisture content of the wood is in equilibrium with the air humidity. The optimum moisture may not be achievable through air drying alone.

After air drying the lumber may be kiln dried. A kiln is a large oven where all variables can be closely monitored. Drying temperatures in a kiln range from 20°C to 50°C (70°F to 120°F), typically requiring 4 to 10 days. Care must be taken to slowly reduce the moisture content of wood. Drying too rapidly can result in cracking and warping. Kiln-dried lumber will take on moisture again if exposed to water; therefore, care must be used when storing and transporting wood.

10.5 Lumber Grades

The final step in wood production involves grading the lumber according to quality. Typically, lumber is graded according to the number of flaws that affect strength, durability, or workability. The most common grade-reducing qualities of lumber are knots, checks, pitch pockets, shakes, and stains. Due to the high degree of natural variability within lumber, it is nearly impossible to develop an exact, uniform set of grading standards. As a result, grading techniques and standards can, and do, vary among organizations. Organizations such as the National Bureau of Standards and the United States Department of Agriculture have spent many years trying to develop a simple, uniform method of lumber sizing, common nomenclature, and grading standards.

The following agencies are certified by the American Lumber Standards Committee Board of Review (Germantown, MD) for inspecting and grading of untreated lumber:

- Northeastern Timber Manufacturer Association (NELMA), Falmouth, MN
- Northern Hardwood and Pine Manufacturer Association (NHPMA), Green Bay, WI
- Redwood Inspection Service (RIS), San Francisco, CA
- Southern Pine Inspection Bureau (SPIB), Pensacola, FL
- West Coast Lumber Inspection Bureau (WCLIB), Portland, OR
- Western Wood Products Association (WWPA), Portland, OR
- National Lumber Grader Authority (NLGA), Ganges, B.C., Canada

Each of these agencies writes standards and specifications for particular species or combinations of species that are produced within their operating region. For example, the SPIB provides the grading rules for all species of Southern Pine, the WWPA governs grading of Ponderosa Pine, and the NLGA provides grading rules for all grades produced in Canada.

10.5.1 ■ Hardwood Grades

The National Hardwood Lumber Association bases the grading of hardwood on the amount of usable lumber in each piece of standard length lumber. The inspection is performed on the poorest side of the material and the grade is based on the number and size of clear “cuttings” that can be produced from a given piece of lumber. Cuttings must have one face clear of strength-reducing imperfections and the other side must be sound. Based on “cutting” quantity, the wood is given a classification of *Firsts*, *Seconds*, *Selects*, and *Common* (No. 1, No. 2, No. 3A, or 3B), with Firsts being the best. Frequently, Firsts and Seconds are grouped into one grade of Firsts and Seconds, FAS.

10.5.2 ■ Softwood Grades

Softwood for structural applications is either graded by visual inspection or is machine stress graded. The purpose of grading is to ensure that all lumber graded to a specific grade designation has at least the minimum mechanical or load-carrying capability with respect to critical design parameters. Under machine stress grading, each piece of wood is subjected to a bending stress, then based on the mechanical response, the wood is graded, as shown in Table 10.3. The grade designation identifies the minimum extreme fiber bending stress, tensile stress parallel to grain, compressive stress parallel to grain, and modulus of elasticity of the wood.

For visual classification, the basic mechanical properties of the wood are determined by testing small clear wood specimens. These results are then adjusted for allowable defects and characteristics for each class of wood. Unlike stress graded lumber, visual stress grade properties are defined for each species of softwood. Table 10.4 is an example of design values for grades of Eastern White Pine.

Visual grade designations include *Yard*, *Structural*, or *Factory and Shop*, with subgrades of *Select*, *Select B*, *Select C*, and *No. 1*, *No. 2*, and *No. 3* commons, appearance, and studs. Other commonly used ratings are *Construction*, *Standard*, or *Utility* and combinations such as *No. 2&BTR* (Number 2 and Better) and *STD&BTR*. Not all grades are used for all species of wood.

Yard lumber frequently refers to some specialty stress grades of lumber such as those used for light structural framing. Structural lumber typically comprises pieces 50 mm to 125 mm (2 in. to 5 in.) thick and is graded according to its intended use. Grading categories include light framing, joists and planks, beams and stringers, and posts and timbers. Factory and shop lumber includes siding, flooring, casing, shingles, shakes, and finish lumber.

TABLE 10.3 Sample of Stress Grading of Softwood for Structural Applications^{1,4}

Grade Designation	Design Values, ² psi				
	Bending ³	Tension Parallel to Grain	Compression Parallel to Grain	Modulus of Elasticity	Minimum Modulus of Elasticity
900f-1.0E	900	350	1,050	1,000,000	510,000
1650f-1.3E	1,650	1,020	1,700	1,300,000	660,000
1950f-1.5E	1,950	1,375	1,800	1,500,000	760,000
2250f-1.7E	2,250	1,750	1,925	1,700,000	860,000
2400f-2.0E	2,400	1,925	1,975	2,000,000	1,020,000
2850f-2.3E	2,850	2,300	2,150	2,300,000	1,170,000
3000f-2.4E	3,000	2,400	2,200	2,400,000	1,220,000

¹Courtesy of American Forest & Paper Association, Washington, D.C.

²Stresses apply to lumber used at 19 percent maximum moisture content. When lumber is designed for use where the moisture content will exceed 19 percent for an extended period of time, the values shown herein shall be multiplied by certain *wet service* factors.

³Bending values are applicable to lumber loaded on edge. When loaded flatwise, these values may be increased by multiplying by certain *flat use* factors.

⁴For a complete list of grade designations and more detailed design values see reference (American Forest & Paper Association, 2005).

10.6 Defects in Lumber

Lumber may include defects that affect either its appearance, mechanical properties, or both. These defects can have many causes, such as natural growth of the wood, wood diseases, animal parasites, too rapid seasoning, or faulty processing. Common defect types are shown in Figure 10.8.

Knots are branch bases that have become incorporated into the wood of the tree trunk or another limb. Knots degrade the mechanical properties of lumber, affecting the tensile and flexural strengths.

However, the presence of sound, tight knots may increase the compressive strength, hardness, and shear characteristics of the wood.

Shakes are lengthwise separations in the wood occurring between annual rings. They develop prior to cutting the lumber and could be due to heavy winds.

Wane is bark or other soft material left on the edge of the board.

Sap Streak is a heavy accumulation of sap in the fibers of the wood, which produces a distinctive streak in color.

TABLE 10.4 Example of Design Values of Eastern White Pine^{1,4}

Grade Designation	Size Classification	Design Values, ² psi						
		Bending ³	Tension Parallel to Grain	Shear Parallel to Grain	Compression perpendicular to Grain	Compression Parallel to Grain	Modulus of Elasticity	Minimum Modulus of Elasticity
Select Structural		1,250	575	135	350	1,200	1,200,000	440,000
No. 1		775	350	135	350	1,000	1,100,000	400,000
No. 2	2" & wider	575	275	135	350	825	1,100,000	400,000
No. 3		350	150	135	350	475	900,000	330,000
Stud	2" & wider	450	200	135	350	525	900,000	330,000
Construction		675	300	135	350	1,050	1,000,000	370,000
Standard	2"-4" wide	375	175	135	350	850	900,000	330,000
Utility		175	75	135	350	550	800,000	290,000

¹ Courtesy of American Forest & Paper Association, Washington, D.C.

² Stresses apply to lumber used at 19 percent maximum moisture content. When lumber is designed for use where the moisture content will exceed 19 percent for an extended period of time, the values shown herein shall be multiplied by certain *wet service* factors.

³ Bending values are applicable to lumber loaded on edge. When loaded flatwise, these values may be increased by multiplying by certain *flat use* factors.

⁴ For a complete list of grade designations and more detailed design values see reference (American Forest & Paper Association, 2005).

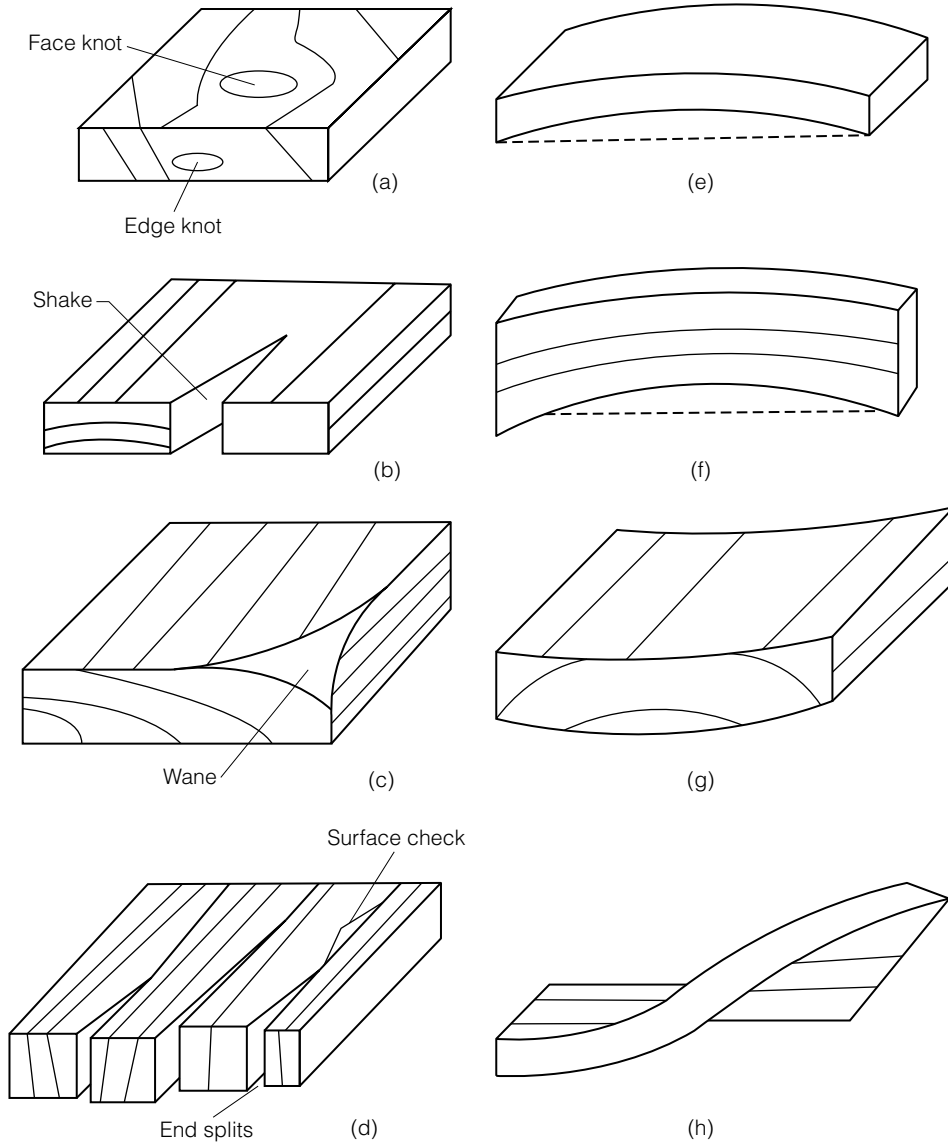


FIGURE 10.8 Common defects in lumber: (a) knots, (b) shakes, (c) wanes, (d) checks and splits, (e) bowing, (f) crooking, (g) cupping, and (h) twisting.

Reaction Wood is abnormally woody tissue that forms in crooked stems or limbs. Reaction wood causes the pith to be off center from the neutral axis of the tree. It creates internal stresses which can cause warping and longitudinal cracking.

Pitch Pockets are well-defined openings between annual rings that contain free resin. Normally, only Douglas fir, pines, spruces, and western larches have pitch pockets.

Bark Pockets are small patches of bark embedded in the wood. These pockets form as a result of an injury to the tree, causing death to a small area of the cambium. The surrounding tree continues to grow, eventually covering the dead area with a new cambium layer.

Checks are ruptures in wood along the grain that develop during seasoning. They can occur on the surface or end of a board. Surface checking results from the separation of the thinner-walled early wood cells and is confined mostly to planer surfaces. Cracks due to end checking normally follow the grain and result in end splitting.

Splits are lengthwise separations of the wood caused by either mishandling or seasoning.

Warping is a distortion of wood from the desired true plane. The four major types of warping are *bowing*, *crooking*, *cupping*, and *twisting*. Bowing is a longitudinal curvature from end to end. Crooking is the longitudinal curvature side to side. Both of these defects result from differential longitudinal shrinkage. Cupping is the rolling of both edges up or down. Twisting is the lifting of one corner out of the plane of the other three. Warping results from differential drying due to the production environment or from the release of internal tree stress.

Raised, Loosened, or Fuzzy Grain may occur during cutting and dressing of lumber.

Chipped or Torn Grain occurs when pieces of wood are scooped out of the board surface or chipped away by the action of the cutting and planing tools.

Machine Burn is an area that has been darkened by overheating during cutting.

10.7 Physical Properties

Important physical properties include specific gravity and density, thermal properties, and electrical properties.

10.7.1 Specific Gravity and Density

Specific gravity of wood depends on cell size, cell wall thickness, and number and types of cells. Regardless of species, the substance composing the cell walls has a specific gravity of 1.5. Because of this consistency, specific gravity is an excellent index for the amount of substance a dry piece of wood actually contains and is nearly constant within each species. Therefore, specific

gravity, or density, is a commonly cited property and is an indicator of mechanical properties within a clear, straight-grained wood.

The dry density of wood ranges from 160 kg/m³ (10 pcf) for balsa to 1000 kg/m³ (65 pcf) for some species. The majority of wood types have densities in the range of 300 to 700 kg/m³ (20 to 45 pcf). Within common domestic species, density may vary by $\pm 10\%$.

10.7.2 ■ Thermal Properties

Thermal conductivity, specific heat, thermal diffusivity, and coefficient of thermal expansion are the four significant thermal properties of wood.

Thermal Conductivity Thermal conductivity is a measure of the rate at which heat flows through a material. The reciprocal of thermal conductivity is the thermal resistance (insulating) value (R). Wood has a thermal conductivity that is a fraction of that of most metals and three to four times greater than common insulating materials. The thermal conductivity ranges from 0.06 W/(m²K) [0.34 Btu/(h-ft²-°F)] for balsa to 0.17 W/(m²K) [1.16 Btu/(h-ft²-°F)] for rock elm. Structural woods average 0.12 W/(m²K) [0.07 Btu/(h-ft²-°F)] as compared to 200 W/(m²K) [115 Btu/(h-ft²-°F)] for aluminum and 0.04 W/(m²K) [0.025 Btu/(h-ft²-°F)] for wool. The thermal conductivity of wood depends on several items including (1) grain orientation, (2) moisture content, (3) specific gravity, (4) extractive content, and (5) structural irregularities such as knots.

Heat flow in wood across the radial and tangential directions (with respect to the growth rings) is nearly uniform. However, heat flow through wood in the longitudinal direction (parallel to the grain) is 2.0 to 2.8 times greater than in the radial direction.

Moisture content has a strong influence on thermal conductivity. When the wood is dry, the cells are filled with air and the thermal conductivity is very low. As the moisture content increases, thermal conductivity increases. As the moisture content increases from 0% to 40%, the thermal conductivity increases by about 30%.

Because of the solid cell wall material in heavy woods, they conduct heat faster than light woods. This relationship between specific gravity and thermal conductivity for wood is linear. Also affecting the heat transfer in wood are increases in extractive content and density (i.e., knots) which increase thermal conductivity.

Specific Heat Specific heat of a material is the ratio of the quantity of heat required to raise the temperature of the material one degree to that required to raise the temperature of an equal mass of water one degree. Temperature and moisture content largely control the specific heat of wood, with species and density having little to no effect. When wood contains water, the specific heat is increased because the specific heat of water is higher than that of dry wood. However, the value of specific heat for the wet wood is higher than just the sum of the specific heats for the wood and water combined. This increase

in specific heat beyond the simple sum is due to the wood–water bonds absorbing energy. An increase in temperature increases the energy absorption of wood and results in an increase in the specific heat.

Thermal Diffusivity Thermal diffusivity is a measure of the rate at which a material absorbs heat from its surroundings. The thermal diffusivity for wood is much smaller than that of other common building materials. Generally, wood has a thermal diffusivity value averaging 0.006 mm/sec. (0.00025 in./sec.), compared with steel, which has a thermal diffusivity of 0.5 mm/sec. (0.02 in./sec.). It is because of the low thermal diffusivity that wood does not feel hot or cold to the touch, compared with other materials. The small thermal conductivity, moderate density, and moderate specific heat contribute to the low value of thermal diffusivity in wood.

Coefficient of Thermal Expansion The coefficient of thermal expansion is a measure of dimensional changes caused by a temperature variance. Thermal expansion coefficients for completely dry wood are positive in all directions. For both hard and soft woods, the longitudinal (parallel to the grain) coefficient values range from 0.009 to 0.0014 mm/m/°C (0.0000017 to 0.0000025 in./in./°F). The expansion coefficients are proportional to density and therefore are five to ten times greater across the grain than those parallel to it.

When moist wood is heated, it expands due to thermal expansion and then shrinks because of the loss of moisture (below the fiber saturation point). This combined swelling and shrinking often results in a net shrinkage. Most woods, at normal moisture levels, react in this way.

10.7.3 ■ Electrical Properties

Air-dry wood is a good electrical insulator. As the moisture content of the wood increases, the resistivity decreases by a factor of three for each 1% change in moisture content. However, when wood reaches the fiber saturation point, it takes on the resistivity of water alone.

10.8 Mechanical Properties

Knowing the mechanical properties of wood is a prerequisite to a proper design of a wood structure. Typical mechanical properties of interest to civil and construction engineers include modulus of elasticity, strength properties, creep, and damping capacity.

10.8.1 ■ Modulus of Elasticity

The typical stress–strain relation of wood is linear up to a certain limit, followed by a small nonlinear curve after which failure occurs, as shown on

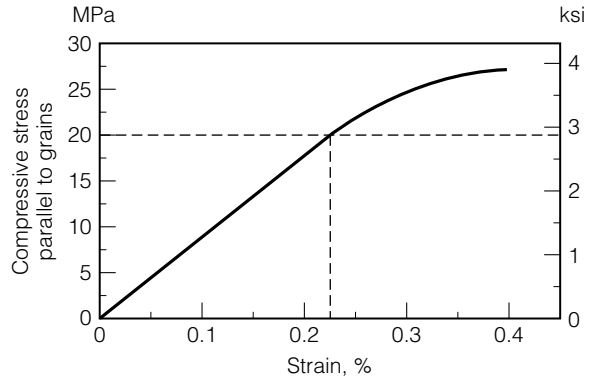


FIGURE 10.9 Typical stress strain relationship for wood.

Figure 10.9. The modulus of elasticity of wood is the slope of the linear portion of the representative stress–strain curve. The stress–strain relation of wood varies within and between species and is affected by variation in moisture content and specific gravity. Also, since wood is anisotropic, different stress–strain relations exist for different directions. The moduli of elasticity along the longitudinal, radial, and tangential axes are typically different.

10.8.2 ■ Strength Properties

Strength properties of wood vary to a large extent, depending on the orientation of grain relative to the direction of force. For example, the tensile strength in the longitudinal direction (parallel to grain) is more than 20 times the tensile strength in the radial direction (perpendicular to grain). Also, tensile strength in the longitudinal direction is larger than the compressive strength in the same direction. Common strength properties for wood include modulus of rupture in bending, compressive strength parallel and perpendicular to the grain, and shear strength parallel to the grain. Some of the less common strength properties are tensile strength parallel to the grain, torsion, toughness, fatigue strength, and rolling shear strength.

10.8.3 ■ Creep

Under sustained loads wood continues to deform or creep. The design values of material properties contemplate fully stressing the member to the tabulated design values for a period of 10 years and/or the application of 90% of the full maximum load continuously throughout the life of the structure. If the maximum stress levels are exceeded, the structure can deform prematurely.

10.8.4 ■ Damping Capacity

Damping is the phenomenon in which the amplitude of vibration in a material decreases with time. Reduction in amplitude is due to internal friction within the material and resistance of the support system. Moisture content

and temperature largely govern the internal friction in wood. At normal ambient temperatures, an increase in moisture content produces a proportional increase in internal friction up to the fiber-saturation point. Under normal conditions of temperature and moisture content, the internal friction in wood (parallel to the grain) is 10 times that of structural metals. Because of these qualities, wood structures dampen vibrations more quickly than metal structures of similar design.

10.9 Testing to Determine Mechanical Properties

Standard mechanical testing methods for wood are designed almost exclusively to obtain data for predicting performance. To achieve reproducibility in the testing environment, specifications include methods of material selection and preparation, testing equipment and techniques, and computational methods for data reduction. Standards for testing wood and wood composites are published by ASTM, the U.S. Department of Commerce, the National Standard Institute (NSI), and various other trade associations, such as the Western Woods Product Association.

Due to the many variables affecting the test results, it is of primary importance to correctly select the specimen and type of test. There are two main testing techniques for establishing strength parameters: the testing of representative, small, clear specimens and the testing of timbers of structural sizes.

The primary purposes for testing small, clear specimens are to obtain the mechanical properties of various species and to provide a means of control and comparison in production activities. The testing of structural timbers provides relationships among mechanical and physical properties, working stress data, correlations between environmental conditions, wood imperfections, and mechanical properties. ASTM D 143 presents the complete testing standards for small, clear wood specimens. This standard gives full descriptions of sample collection, preparation, and testing techniques. Mechanical tests included in this standard are the following:

- Static bending
- Impact bending
- Compression perpendicular to the grain
- Shear parallel to the grain
- Tension parallel to the grain
- Nail withdrawal
- Radial and tangential shrinkage
- Compression parallel to the grain
- Toughness
- Hardness
- Cleavage
- Tension perpendicular to the grain
- Specific gravity and shrinkage in volume
- Moisture determination.

Figure 10.10 shows a schematic of test specimens of wood tested in tension, compression, bending, and hardness. Static and impact bending, compression

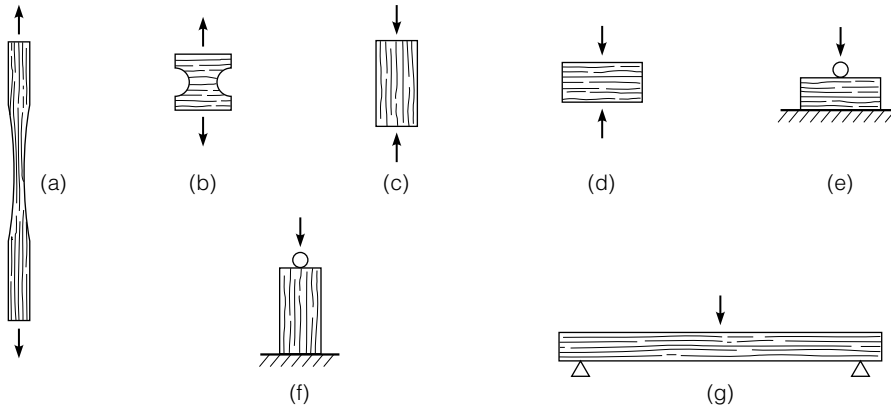


FIGURE 10.10 Test specimens of wood: (a) tension parallel to grains, (b) tension perpendicular to grains, (c) compression parallel to grains, (d) compression perpendicular to grains, (e) hardness perpendicular to grains, (f) hardness parallel to grains, and (g) bending. (© Pearson Education, Inc. Used by permission.)

and tension parallel and perpendicular to the grain, and shear parallel to the grain are commonly used.

10.9.1 ■ Static Bending Test

The static bending test is performed on either $50 \times 50 \times 760$ mm ($2 \times 2 \times 30$ in.) or $25 \times 25 \times 410$ mm ($1 \times 1 \times 16$ in.) specimens. For the large specimens, the loading head is placed on the center of the specimen and over a span of 710 mm (28 in.), and the load is applied at a rate of 2.5 mm/min. (0.1 in./min.). For the small specimens, the loading head is placed on the center of the specimen and over a span of 360 mm (14 in.), and the load is applied at a rate of 1.3 mm/min. (0.05 in./min.). Load–deflection data are recorded to or beyond the maximum load. Within the proportional limit, readings are taken to the nearest 0.02 mm (0.001 in.). After the proportional limit, deflection readings are usually measured with a dial gage, to the limit of the gage, usually 25 mm (1 in.). Load and deflection of the first failure, the maximum load, and points of sudden change are recorded. The failure appearance is described as either brash or fibrous. Brash indicates an abrupt failure and fibrous indicates a failure showing splinters.

Sample Problem 10.2

A static bending test was performed on a $50 \times 50 \times 760$ mm wood sample according to ASTM D143 procedure (span between supports = 710 mm). If the maximum load was 2.67 kN, calculate the modulus of rupture.

Solution

$$\text{Modulus of rupture} = \frac{Mc}{I}$$

where

M = bending moment at maximum load

c = 1/2 of the specimen height

I = moment of inertia of the specimen cross section

$$\text{Reaction at each support at failure} = \frac{2.67}{2} = 1.335 \text{ kN}$$

$$\text{Bending moment at the center at failure} = 1.335 \times \left(\frac{710}{2}\right) = 473.9 \text{ N}\cdot\text{m}$$

$$I = \frac{(0.05)(0.05)^3}{12} = 5.21 \times 10^{-7} \text{ m}^4$$

$$\text{Modulus of rupture} = \frac{473.9 \times 0.025}{5.21 \times 10^{-7}} = 22.75 \text{ MPa}$$

10.9.2 ■ Compression Tests

The compression test parallel to the grain is performed on either $50 \times 50 \times 200 \text{ mm}$ ($2 \times 2 \times 8 \text{ in.}$) or $25 \times 25 \times 100 \text{ mm}$ ($1 \times 1 \times 4 \text{ in.}$) specimens, as shown in Figure 10.11. The load is applied at a rate equal to 0.003 mm/mm

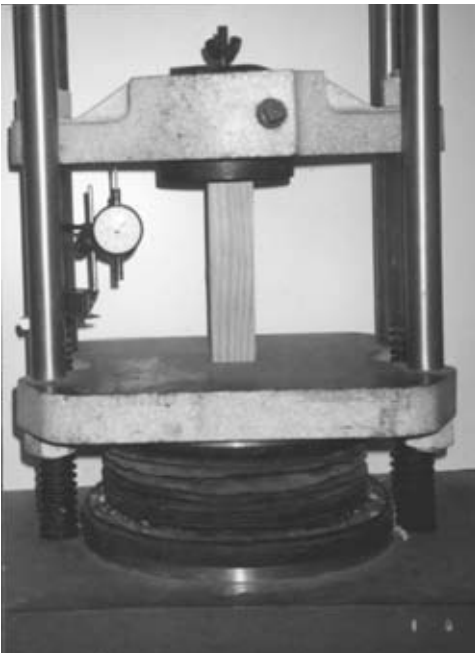


FIGURE 10.11 Compression parallel to grain test.

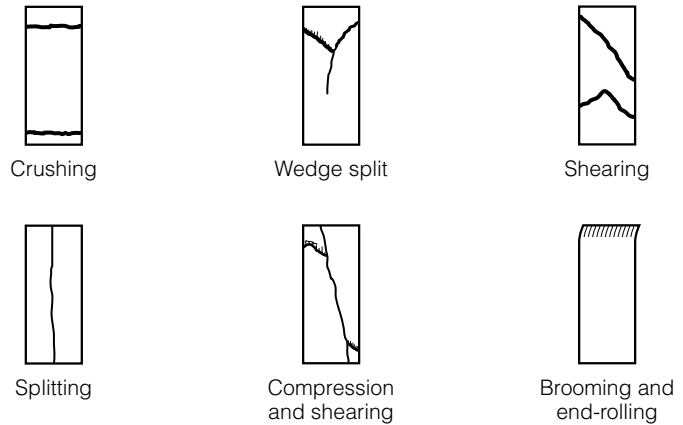


FIGURE 10.12 Types of failure in the compression-parallel-to-grain test. (ASTM D143). Reprinted with permission of ASTM.

(in./in.) of the nominal specimen length per minute. The deformations are recorded to 0.002 mm (0.0001 in.) over a gage length of not more than 150 mm (6 in.) for the large specimens or 50 mm (2 in.) for the small specimens. Load-compression readings are recorded until well past the proportional limit. The failures should occur in the center portion of the sample. If failures are occurring near the ends, the samples can be stacked such that the ends dry relative to the middle. This will increase the strength of the ends of the sample. The tests are then repeated on the conditional samples. The type of failure can be classified as crushing, wedge split, shearing, splitting, compression and shearing, and brooming and end rolling, as shown in Figure 10.12.

The compression test perpendicular to the grain is performed on $50 \times 50 \times 150$ mm ($2 \times 2 \times 6$ in.) specimens. The load is applied through a metal bearing plate 50 mm (2 in.) in width, centered across the upper surface of the specimen. The load is applied at a rate of 0.305 mm/min. (0.012 in./min.). Deflection readings are taken to the nearest 0.002 mm (0.0001 in.). Load and deformation are measured until the deformation is 2.5 mm (0.1 in.).

10.10 Design Considerations

Measurement of wood properties in the lab does not reflect all the factors which affect behavior of the material in engineering applications. For design of wood structures, the strength properties given in Tables 10.3 and 10.4 must be adjusted for the following (National Design Specification[®] for Wood Construction, 2005, ASTM D2555):

- Wet service
- Temperature
- Beam stability
- Size

- Volume
- Flat use
- Repetitive member
- Curvature
- Form
- Column stability
- Shear stress
- Bearing area

In addition, sustained loads will cause wood to creep. In design applications, this means that the wood can carry higher stresses of short time durations than the same wood element can carry for a long-term application. Generally, a load duration of 10 years is used for design. Typical load duration factors that would be applied to the values in Tables 10.3 and 10.4 (except for compression perpendicular to the grain) are shown in Figure 10.13. For example, if a floor joint is designed for a temporary stage that is used only for a one-day presentation, the allowable bending fiber stress can be increased by 33 percent over the allowable stress for a normal application.

10.11 Organisms that Degrade Wood

Wood can experience degradation due to attack of fungi, bacteria, insects, or marine organisms.

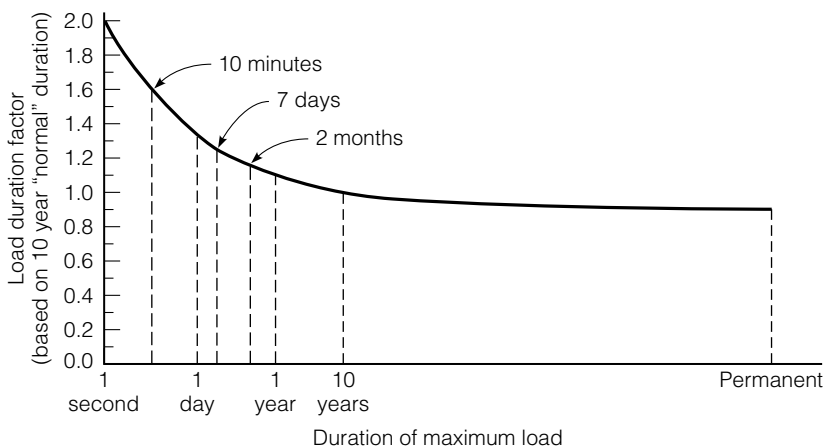


FIGURE 10.13 Adjustment factors for load duration working-stress for wood design based on 10-year nominal duration. Reprinted with permission of American Forest and Paper Association (2005.)

10.11.1 ■ Fungi

Most forms of decay and sap stains are the result of fungal growth. Fungi need four essential conditions to exist: food, proper range of temperature, moisture, and oxygen. Fungi feed on either the cell structure or the cell contents of woody plants, depending on the fungus type. The temperature range conducive for fungal growth is from 5°C to 40°C (40°F to 100°F). Moisture content above the fiber saturation point is required for fungal growth. Fungi are plants and, as such, require oxygen for respiration. Fungi attack produces *stains* and/or *decay damage*.

To protect against fungal attack, one of the four essential conditions for growth needs be removed. The most effective protection measure is to keep the wood dry by using coatings or by correct placement during storage and in the structure. Fungi can also be contained by treating the wood fibers with chemical poisons through a pressure treatment process.

Construction procedures that limit decay in buildings include the following:

1. Building with dry lumber that is free of incipient decay and excessive amounts of stains and molds
2. Using designs that keep the wood components dry, using wood treated with preservatives
3. Using a heartwood from decay-resistant species in sections exposed to above-ground decay hazards
4. Using pressure-treated wood for components in contact with the ground.

10.11.2 ■ Bacteria

Bacteria causes “wet wood” and “black heartwood” in living trees and a general degradation of lumber. Wet wood is a water-soaked condition that occupies the stem centers of living trees and is most common in poplar, willows, and elms. Black heartwood has characteristics similar to those of wet wood, in addition to causing the center of the stem to turn dark brown or black.

Bacterial growth is sometimes fostered by prolonged storage in contact with soils. This type of bacteria activity produces a softening of the outer wood layers, which results in excessive shrinkage when redried.

10.11.3 ■ Insects

Beetles and *termites* are the most common wood-attacking insects. Several types of beetles, such as bark beetles, attack and destroy wood. Storage of the logs in water or a water spray prevents the parent beetle from boring. Quick drying or early removal of the bark also prevents activity by beetle attack. Damage can be prevented by proper cutting practices and dipping or spraying with an appropriate chemical solution.

Termites are perhaps the most destructive organism that attacks wood. The annual damage attributed to termites exceeds losses due to fires. Termites enter structures through wood that is close to the ground and is poorly ventilated or wet. Prevention is achieved by painting and otherwise prohibiting insect entry into areas of unprotected wood through the use of screening, sill plates, and sealing compounds.

10.11.4 Marine Organisms

Damage by marine boring organisms in the United States and surrounding oceans is principally caused by shipworms, pholads, *Limnoria*, and *Sphaeroma*. These organisms are almost totally confined to salt or brackish waters.

10.12 Wood Preservation

Paints, petroleum-based solutions, and waterborne oxides (salts) are the principle types of wood preservatives. The degree of preservation achieved depends on the type of preservative, the degree of penetration, and the amount of the chemical retained within the wood. Paints are applied on the surface, while the other preservatives are applied under pressure to increase penetration into the wood.

10.12.1 Petroleum-Based Solutions

Coal-tar creosote, petroleum creosote, creosote solutions, and pentachlorophenol solutions are the oil-based preservatives. These preservatives are very effective, but are environmentally sensitive. They are commonly used where a high degree of environmental exposure exists and human contact is not a concern. Applications include utility poles, railroad ties, and retaining walls.

10.12.2 Waterborne Preservatives

The typical solutes used in waterborne preservative mixtures are ammoniacal copper arsenate, chromated copper arsenate (CCA) and ammoniacal copper zinc arsenate. The advantages of the waterborne preservative over the oil-based are cleanliness and its ability to be painted. The disadvantage of these treatments is their removal by leaching when exposed to moist conditions over long periods of time. These preservatives are also environmentally sensitive and must be applied under carefully controlled conditions. The level of potential danger to humans from contacting wood pressure treated with CCA is controversial. Trade groups are supportive of the product, but several other agencies see potential health effects. By 2003, the wood preservation industry agreed to stop using arsenic-based preservatives for products intended for residential use or direct human contact. CCA can still be

used for commercial applications and permanent foundations. Although CCA was removed from the market, the EPA is not calling for the removal of it from existing structures.

The most common replacement preservatives are ammoniacal copper quat, or ACQ, followed by copper azole, and borate. Borate may be used in home foundation sill plates and other “dry” applications, but borate-treated wood is not appropriate for outdoor uses (Forintek, 2002).

10.12.3 Application Techniques

Preservatives are applied by superficial treatment or by fluid penetration processes. Superficial treatment techniques include coatings applied by painting, spraying, or immersion. Liquid penetration into a porous solid is by capillary action and is a function of surface tension, angle of contact, time, temperature, and pressure.

Pressure-treated wood has greater resistance to degradation than surface-treated wood. The preservative is forced into the entire structure of the wood. By thoroughly treating the entire cross section of the wood, decay can be eliminated for an extended period of time. Some vendors of pressure-treated wood provide a lifetime warranty for their products when in direct contact with the ground. The key to ensuring long life is the amount of preservative retained in the wood. Table 10.5 gives the minimum retention requirements for different treatments and applications of Southern Pine lumber, timber, and plywood.

TABLE 10.5 Minimum Retention Requirements in kg/m³ for Different Treatments and Applications of Southern Pine Lumber, Timber, and Plywood (Southern Pine Council 1990)

		Above ground	Soil and fresh water use	Permanent wood foundation	Saltwater use
Waterborne preservatives	Ammoniacal copper arsenate	4	6	10	41
	Ammoniacal copper zinc arsenate	4	7	10	41
	Chromated copper arsenate	4	7	10	41
Creosote and oil-borne preservatives	Creosote	131	163	NR*	408
	Creosote–petroleum	131	163	NR*	NR*
	Creosote solutions	131	163	NR*	408
	Pentachlorophenol	7	8	NR*	NR*

*NR: Not recommended.

**For a complete list of wood products see the American Wood Preservative Association (AWPA) standards.

10.12.4 ■ Construction Precautions

There are a few issues that should be recognized during the design and construction with pressure-treated materials. Care should be taken to avoid inhaling saw dust during cutting of the wood. Hands should be washed after handling pressure treated wood and before consuming food or beverages. Clothes should be washed separately from other items. Waste material must be disposed of properly; it should not be burned.

Pressure-treated wood promotes corrosion of hangers and other fasteners. Generally, galvanization was adequate. However, there is some evidence that the new preservatives are even more corrosive. The higher metal corrosion rates associated with ACQ-treated wood have raised concerns with the federal Consumer Product Safety Commission (CPSC) and a Bay Area district attorney who recently issued a consumer alert (e-builder, 2005).

“CPSC is recommending consumers use stainless steel brackets and fasteners in conjunction with ACQ-treated lumber,” said commission spokesman Scott Wolfson. The CPSC is considering whether it needs to study the corrosion issue further, based on information from the connector industry and Contra Costa County. That county’s district attorney, Bob Kochly, warned in a recent consumer alert that wood treated with ACQ and copper azole “may result in serious and premature corrosion ... especially in wet or moist conditions unless stainless steel connectors are used.”

10.13 Engineered Wood Products

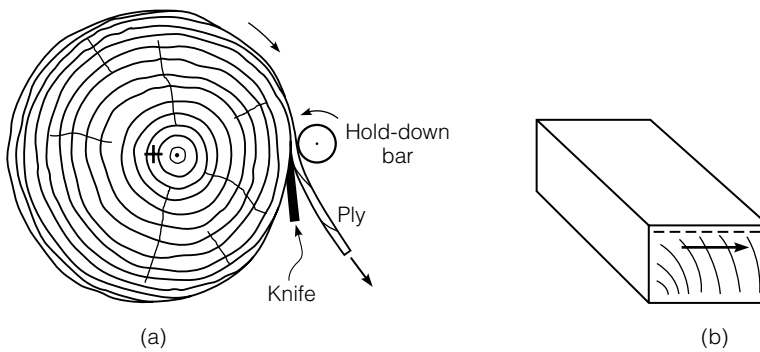
Engineered wood includes a wide variety of products manufactured by bonding together wood strands, veneers, lumber, or other forms of wood fibers to produce large and integral units. These products are “engineered” to produce specific and consistent mechanical behavior and thus have consistent design properties. There is a wide variety of engineered wood products produced for many applications. Only the products used in structural applications are considered here.

An engineered product consists of wood stock material glued together with an appropriate resin or adhesive. These are predominantly wood materials, so they are liable to the same concerns as natural wood products with respect to the effects of fire, moisture, and decay. The wood stock may consist of veneers, strands, or dried lumber as shown in Table 10.6.

Veneer-based materials consist of wood plies that are glued together. A ply is a thin sheet of wood. To produce plies, logs are saturated by storage in ponds, water vats, steam vats, or water sprays. Prior to processing, the logs are moved into a boiling water bath. Next, they are debarked and sectioned into the desired width. The segments are rotated in a giant lathe and peeled into continuous sheets of veneer, or sliced, as shown in Figure 10.14. These segments of veneer are trimmed and combined into continuous rolls. Each roll of veneer is seasoned, dried to the desired moisture content. The plies,

TABLE 10.6 Classification of Engineered Wood Products

Wood stock	Class of Product	
	Structural Panels/Sheets (sheathing, flooring)	Structural Shapes (beams, columns, headers)
Veneer	Plywood	Laminated veneer lumber
Strands	Flakeboard	Oriented strand lumber
	Waferboard - random orientation of strands	Laminated strand lumber
	Oriented Strand Board - strands oriented, panels made in plies	Parallel strand lumber (Parallam)
Composite	COM-PLY - Wood fiber core with veneer exterior	N/A
Dried lumber	N/A	Glued laminated lumber (Glulam)

**FIGURE 10.14** Cutting of plies for plywood and glulam: (a) rotary cutting and (b) slicing (© Pearson Education, Inc. Used by permission.)

cut from the veneer rolls, are assembled, glued, and pressed. The grains of the plies are alternated at 90° for plywood and they are arranged parallel for laminated veneer lumber. Plywood is made in sheets, whereas laminated veneer lumber is made in structural shapes.

Strands have uniform thickness, with a combination of length and width dimensions up to 150 mm (6") long and 25 mm (1") wide for panel products, and may be up to 600 mm (24") long for structural shapes. Plies of strands are made by gluing together the strands to form plies. Structural panels are made by gluing together plies in three or five layers. These

panels fall into a general class of material called flakeboard. Originally, the strands had a random orientation, producing waferboard. In the 1970s, recognition of the advantages of orientating the strands prior to gluing, then forming panels by using plies with alternating orientation of strands, led to the development of oriented strandboard (OSB). This is the dominant strand product used in the market today. It's structural characteristics compare favorably with plywood. Strandboard can also be made into structural shapes, such as beams.

Dried lumber products can be glued together (laminated) to form structural shapes to meet a wide variety of applications. Typically, either southern pine or western species of softwoods are used. The individual boards are referred to as lams, which are glued together to form the glulam product. Lams of southern pine are typically 35 mm (1 3/8 inch); Western species lams are typically 38 mm (1 1/2 inches). The widths of glulam products are typically 63 to 275 mm (2 1/2 to 10 3/4 inches), although virtually any size member can be produced.

Engineered wood products can be broadly classified as either structural panels/sheets or structural shapes as shown in Table 10.6. Panels are primarily used for sheathing, whereas structural shapes are used as beams, columns, etc. Sheet products and structural shape products can be combined to produce a variety of composite structural members, such as I-joists and T-sections.

10.13.1 ■ Structural Panels/Sheets

Structural panel products include plywood, oriented strand board (OSB), and COM-PLY. Plywood is manufactured as a composite of veneer plies, with an alternating orientation of the grain. OSB is manufactured as a composite of strands; the strands are oriented and pressed into sheets. The OSB panel consists of three to five sheets glued together, with alternating orientation of the strands. COM-PLY is a proprietary product manufactured with a wood fiber core and veneer exterior. Panels may be used directly as sheathing, roofs, sides, and floors. Sheet panels can be cut and bonded into structural components, such as glued or nailed box beams, stressed skin panels, and structural insulated panels.

Conventional wood-based composite products are typically made with a thermosetting or heat-curing resin or adhesive that binds the wood fibers together. Adhesives are chosen based on their suitability for the particular product. Factors considered are the materials to be bonded, moisture content at time of bonding, mechanical property, durability, and cost (USDA-FS, 1999). Commonly used resin-binder systems for panels include the following:

Phenol-formaldehyde (PF) Used for materials designed for exterior exposure, including plywood, OSB, and siding.

Urea-formaldehyde (UF) typically used for manufactured products where dimensional uniformity and surface smoothness are of primary concern, such as particle board and medium density fiberboard designed for interior applications.

Melamine–formaldehyde (MF) used for decorative laminates, paper treating, and paper coating.

Isocyanate as diphenylmethane di-isocyanate, commonly used for OSB. Highly toxic during the manufacturing process.

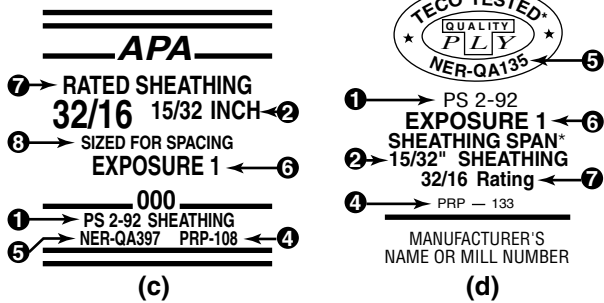
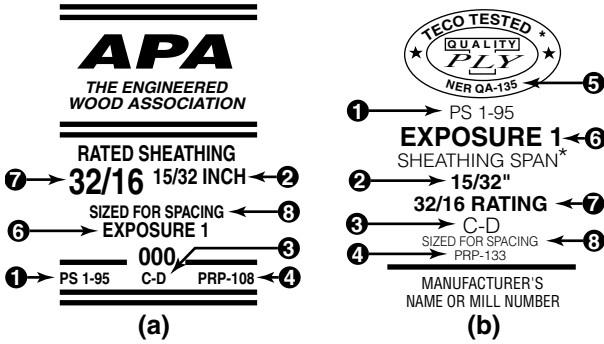
UF and PF systems are expected to continue to be the dominate wood adhesives, unless their use is curtailed due to concerns about the hazards of formaldehyde products or interruptions to the supply and manufacturing of the material. Research is underway to develop adhesives from renewable resources.

Structural panels are manufactured to meet requirements based on either the composition of the material or the stress rating of the panels. The National Institute of Standards and Technology, NIST, publishes these standards. PS 1-95 specifies the composition of structural plywood. This standard specifies wood species, grades of veneer, what repairs are permissible, and how the repairs may be made. NIST standard PS 2-92 is a performance-based standard for structural composite panels. Figure 10.15 shows typical grade stamps for plywood and OSB (USDA-FS, 1999). Figures 10.15 (a) and (b) are grade stamps for plywood showing (1) conformance of plywood to product standards, (2) nominal panel thickness, (3) grades of face and back veneers or grade name based on panel use, (4) performance-rated panel standard, (5) recognition as a quality assurance agency by the National Evaluation Service, NES, (6) exposure durability classification, (7) span rating, which refers to maximum allowable roof support spacing and maximum floor joist spacing, and (8) panel size for spacing. Figure 10.15 (c) and (d) demonstrate the typical grade stamps for OSB. The ratings are similar to those for plywood, with the exception that OSB does not have a grade rating for the face and back veneer material.

The mechanical properties of plywood are rated based on the orientation of the strength axis relative to the orientation of the supports, as shown on Figure 10.16 (Williamson, 2002). The differences in the strength properties are due to the orientation of the grains in the layers of the plywood. Similar layers occur in OSB; however, the layers are neither defined nor specified. The mechanical properties of structural panels are rated based on the orientation of the load, as shown on Figure 10.17, and several other factors (Williamson, 2002). Table 10.7 summarizes the mechanical properties of plywood and OSB used in structural sheathing applications (USDA-FS, 1999). As with natural wood products, there are several design adjustment factors that must be considered in the selection of structural panel for a specific application.

10.13.2 ■ Structural Shapes

Three classes of engineered wood products are used to manufacture structural shapes, based on the type of wood stock: laminated veneer, strand, and glue laminated. The laminated and strand products are classified as structural composite lumber (SCL). All types of SCL products can be used in place



- 1 Product Standard that governs specifics of production for construction and industrial plywood
- 2 Nominal panel thickness subject to acceptable tolerances
- 3 Panel grade designation indicating minimum veneer grade used for panel face and back, or grade name based on panel use
- 4 Performance-rated panel standard indicating structural-use panel test procedure recognized by National Evaluation Service (NES)
- 5 NES report number from Council of American Building Officials (CABO)
- 6 Exposure durability classification: Exposure 1 indicates interior panel bonded with exterior glue suitable for uses not permanently exposed to weather
- 7 Span rating indicating maximum spacing of roof and floor supports for ordinary residential construction applications; 32/16 rating identifies a panel rated for use on roof supports spaced up to 813 mm (32 in.) o.c., or floor supports spaced up to 406 mm (16 in.) o.c.
- 8 Sized for spacing denotes panels that have been sized to allow for spacing of panel edges during installation to reduce the possibility of buckling

FIGURE 10.15 Typical grade stamp for structural panels.

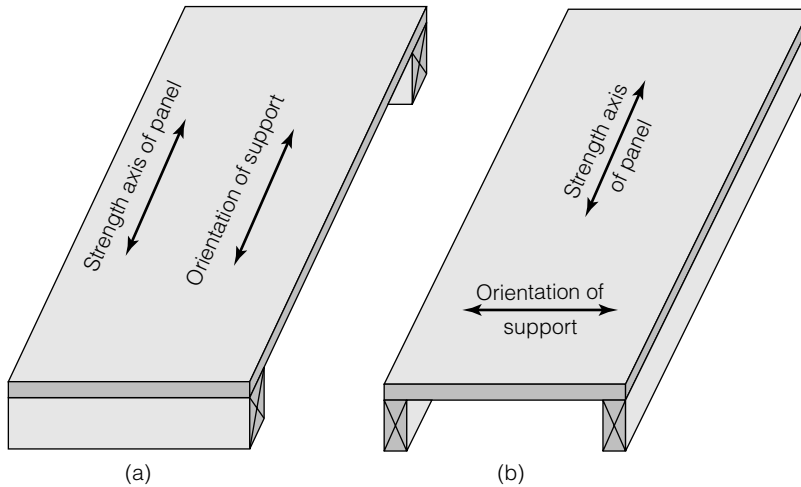


FIGURE 10.16 Strength axis of plywood.

of sawn lumber for many applications. Laminated veneer lumber (LVL) is used for scaffold planks and in flanges of prefabricated I-joists. Both LVL and parallel strand lumber (PSL) are used for headers and major load-carrying elements. The strand products are used for band joists in floor construction and as substitutes for studs and rafters in wall and roof construction (USDA-FS, 1999).

Laminated Veneer Lumber Laminated veneer lumber (LVL) was developed in the 1940s to produce high-strength parts for aircraft. It is currently a widely accepted construction material. Veneer sheets, 2.5 mm to 3.2 mm (1/10 in. to 1/8 in.) thick are glued with phenol–formaldehyde adhesive to form billets, which are 0.6 m to 1.2 m (2 ft to 4 ft) wide by 38 mm (1.5 inches) thick. The veneer sheets are all aligned with the grain parallel to the length of the member. The end joint between sheets is either staggered or overlapped to minimize any strength-reducing effect. Continuous presses are used to form sheets with lengths limited only by handling capability. Alternatively, the billets may be manufactured in 2.4 m (8 ft) lengths and enjoined with either scarf or finger joints to form longer pieces.

The veneer used to manufacture LVL must be carefully selected for uniformity. Visual grading may be used, but ultrasonic testing is commonly used to identify and sort acceptable sheets. The ultrasonic testing can rapidly detect flaws in the veneer, such as knots and splits that would limit the strength of the finished product. Since the quality of each veneer layer is controlled, the variation in product properties are less than for natural products.

LVL is manufactured to meet specific stress grade requirements. Due to the ply construction, the orientation of the member and load must be considered when selecting properties. Figure 10.18 identifies the common

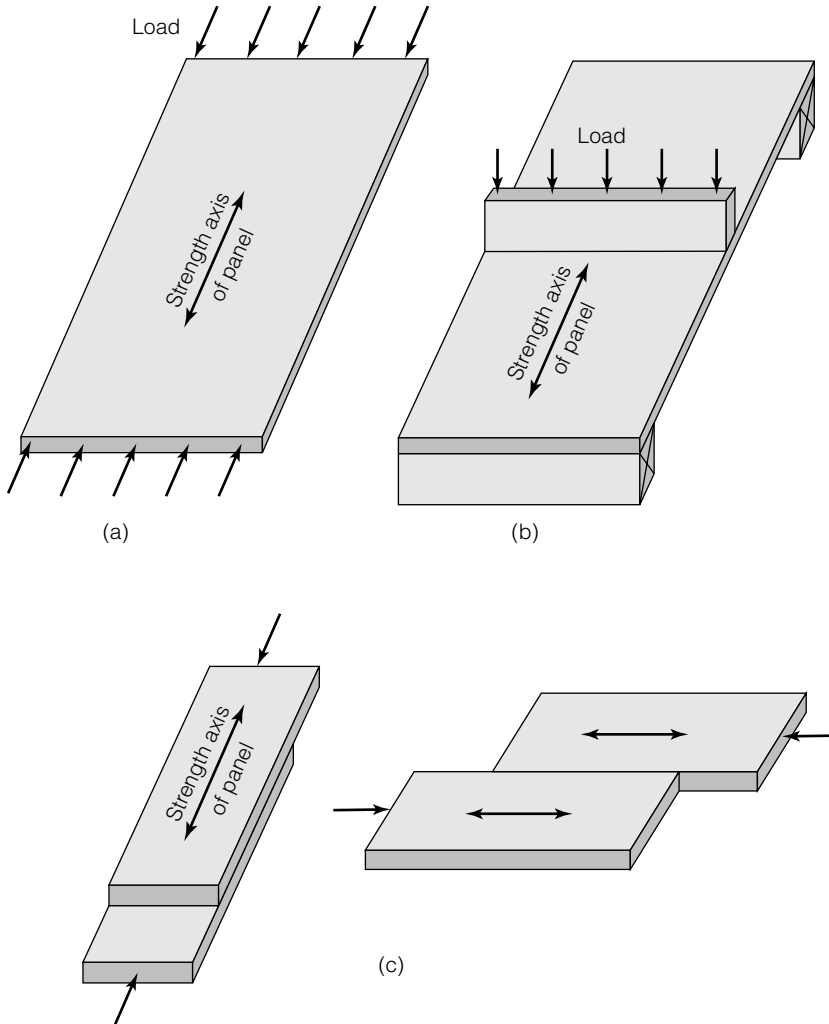


FIGURE 10.17 Load orientation conditions for determining mechanical properties of plywood.

terminology used to describe loading conditions for testing and design (Williamson, 2002). Table 10.8 is a portion of the design property table in the APA-EWS performance standard PRL-501, which covers the required properties for stress-graded LVL (APA-EWS, 2000). There are several footnotes associated with the information in this table; therefore, the cited reference should be consulted before using this information for design. The footnotes deal with modifications to the design values as a function of load conditions and duration. It should be noted that the values in Table 10.8 are for design and are therefore conservative. Table 10.9 provides characteristic test values, which are much higher than the design values for all properties other than elastic modulus in bending.

TABLE 10.7 General Properties of Plywood and OSB in Structural Sheathing Applications.

Property	Plywood	OSB	ASTM Test Method
Linear hygroscopic expansion (30–90%), %	0.15	0.15	
Linear thermal expansion in./in./F	3.4×10^{-6}	3.4×10^{-6}	
Flexure—modulus of rupture, psi	3000–7000	3000–4000	D 3043
Flexure—modulus of elasticity, psi	$1.0-1.9 \times 10^6$	700–1200	
Tensile strength, psi	1500–4000	1000–1500	D 3500
Compressive strength, psi	3000–5000	1500–2500	D 3501
Shear through thickness—strength, psi	600–1100	1000–1500	D 2719
Shear through thickness—modulus, psi	$68-110 \times 10^3$	$180-290 \times 10^3$	D 3044
Shear in plane of plies—strength, psi	250–300	200–300	D 2718
Shear in plane of plies—modulus, psi	$20-30 \times 10^3$	$20-50 \times 10^3$	D 2718

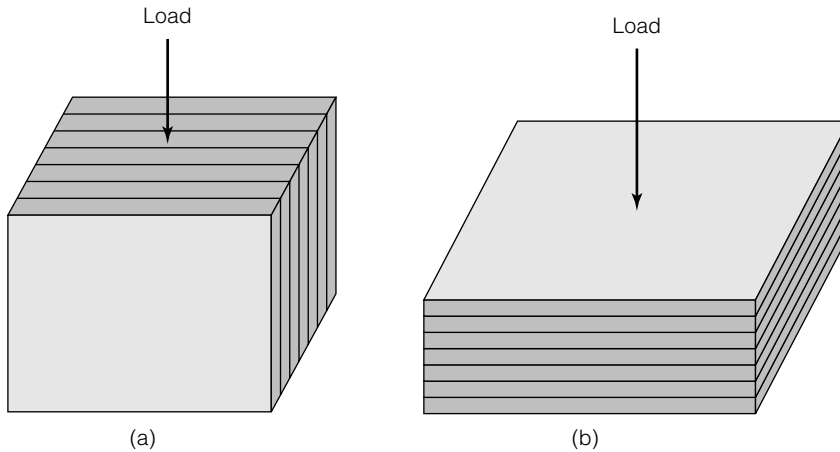


FIGURE 10.18 Axis convention used for defining loading conditions for Laminated Veneer Lumber.

TABLE 10.8 Design Properties for APA-EWS Performance Rated LVL (APA-EWS, 2000)

APA-EWS Stress Class	Bending modulus (10^6 psi)	Allowable extreme fiber stress (psi)	Allowable tensile stress parallel to grain (psi)	Compressive stress parallel to grain (psi)	Edgewise shear stress (psi)	Stress perpendicular to grain (psi)
1.5E-2250F	1.5	2250	1500	1950	220	575
1.8E-2600F	1.8	2600	1700	5400	285	700
1.9E-2600F	1.9	2600	1700	2550	285	700
2.0F-2900F	2.0	2900	1900	2750	285	750
2.1E-3100F	2.1	3100	2200	3000	285	850

Parallel and Oriented Strand Lumber As with OSB, strands of wood can be glued together into structural members. Depending on the length of strand and degree of orientation, products are classified as parallel strand lumber, laminated strand lumber, or oriented strand lumber.

Parallel strand lumber is a composite of wood strand elements, with wood fibers oriented primarily along the length of the member. The smallest dimension of the strand must be less than 6.4 mm (0.25 in.) and the average length of the strand must be 150 times the smallest dimension. Typically, the strands are 3 mm (0.125 in.) thick, 19 mm (0.75 in.) wide and 0.6 m (24 in.) long. A waterproof structural adhesive, such as phenol-resorcinol-formaldehyde,

TABLE 10.9 Characteristic Test Values for APA-EWS Performance Rated LVL (APA-EWS, 2000)

APA-EWS Stress Class	Bending modulus (10 ⁶ psi)	Allowable extreme fiber stress (psi)	Allowable tensile stress parallel to grain (psi)	Compressive stress parallel to grain (psi)	Edgewise shear stress (psi)	Compressive stress perpendicular to grain (psi)
1.5E-2250F	1.5	4725	3150	3705	695	960
1.8E-2600F	1.8	5460	3570	4560	900	1170
1.9E-2600F	1.9	5460	3570	4845	900	1170
2.0F-2900F	2.0	5460	3990	5225	900	1255
2.1E-3100F	2.1	5460	4620	5700	900	1420

bonds the strands together. The strands are carefully oriented, coated with adhesive, and run through a continuous press to achieve a high density, producing billets that are typically 0.28 m by 0.48 m (11 in. by 19 in.). Length is limited only by handling capability. The billets are sawn into the desired dimensions (USDA-FS, 1999).

Currently, parallel strand lumber is a proprietary material with a single producer, marketed under the trade name Parallam (Trusjoist, 2003). The mechanical design properties of this product, as reported by the manufacturer, are given in Table 10.10.

TABLE 10.10 Mechanical Properties of Parallel Strand and Laminated Strand Lumber (Trusjoist)

Property	Parallel Strand Lumber	Laminated Strand Lumber
Shear modulus of elasticity (psi)	125,000	
Modulus of elasticity (psi)	2.0×10^6	1.5×10^6
Flexural stress (psi)	2900	2250
Tension stress (psi)	2025	
Compression perpendicular to grain (psi)	750	475
Compression parallel to grain (psi)	2900	1950
Horizontal shear parallel to grain (psi)	290	400

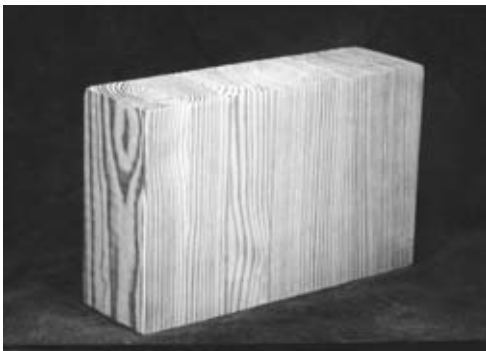
Laminated strand lumber (LSL) and oriented strand lumber (OSL) are extensions of the technology used to produce oriented strand board. They are manufactured using similar processes, with the primary difference being in the length of the strands—approximately 300 mm (12 in.) and 150 mm (6 in.) for LSL and OSL, respectively. As shown in Table 10.10, the mechanical properties of LSL are somewhat less than those of PSL. The mechanical properties of OSL are less than those of LSL (USDA-FS, 1999).

Glued-Laminated Timbers Laminated timbers are composed of two or more layers of dimensional lumber glued together with the grain of all layers laid parallel. Almost any species can be used when its mechanical and physical properties are suited for the design requirements. However, softwoods, such as Douglas fir and southern pine, are most commonly used for laminated structural timbers.

Glued-laminated wood, commonly called “glulam,” is used for structural beams and columns, furniture, sports equipment, and decorative wood finishes, as shown in Figure 10.19. In such cases, glued-laminated wood is preferred over large one-piece members for many reasons, including the following:

- ease of manufacturing of large structural members from standard commercial sized lumber
- the opportunity to design large members that vary in cross section along their length, as required by the application
- specialized design to meet architectural appeal, opportunity to use lower grades of wood within the less stressed areas of the member
- minimization of checking and other seasoning defects associated with curing large one-piece members

Assembly of the laminates into full-depth members is a critical stage in manufacturing. Laminations are planed to strict tolerances just before gluing to ensure the final assembly will be rectangular and that pressure



(a)



(b)

FIGURE 10.19 Examples of glued-laminated wood.

will be applied evenly. Prequalified adhesives, typically phenol–resorcinol, are applied to the gluing face. The laminations are laid up and clamped as an assembly while the glue cures. The adhesive is allowed to cure at room temperature for 6 to 24 hours, or, in some newer processes, radio-frequency curing is used to reduce the time to a few minutes (USDA-FS, 1999).

Glued–laminated wood is designed and produced to have consistent and uniform properties. Due to the laminated structure, the glulam can be laid up to optimize the use of the wood products. For example, glulams designed as bending members are laid up with higher quality wood at the edges of the beam and lower quality wood in the center, as shown in Figure 10.20. Bending members used for cantilevers or continuous members over multiple spans have a balanced layup, since both the top and bottom of the beam carry tensile loads. However, glulams designed for simple spans, which are designed to have only tension at the bottom and compression at the top of the beam, have unbalanced designs, with the higher quality wood at the bottom of the member. This is an unbalanced design and these members are carefully labeled to minimize the risk of installing the beam in an incorrect orientation.

Glulams are produced in four appearance grades:

Industrial grade is suitable where the glulam will not be exposed to view and appearance is not a concern.

Framing grade members match the width of conventional framing for use as door and window headers.

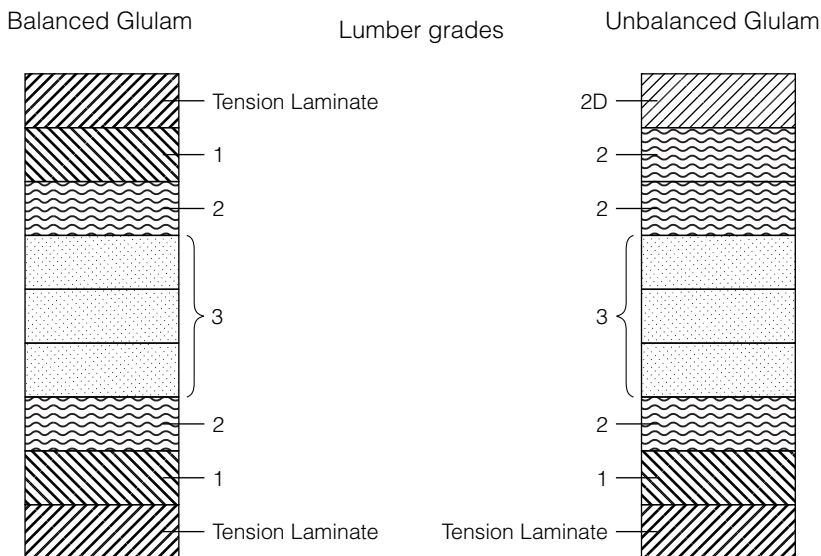


FIGURE 10.20 Lay up arrangement for balanced and unbalanced glulams.

Architectural grade is suitable where appearance is an important requirement.

Premium grade is the highest grade and is used when appearance is of the utmost importance.

Appearance grades do not modify design values, grades of lumber used, or other provisions governing the manufacture or use of glued laminate timber.

Glulams are specified by stress class, based on minimum values for their flexural strength and elastic modulus, as shown in Table 10.11 (APA-EWS, 2004). The design values in this table are for members with four or more laminations. The stress grade classification does not designate the species used

TABLE 10.11 Stress Classes of Glulams (APA-EWS, 2004)

Extreme Fiber in Bending					
Stress Class	Tension Zone Stressed in Tension (psi)	Compression Zone Stressed in Tension¹, (psi)	Compression Perpendicular to grain (psi)	Shear Parallel to Grain³ (psi)	Modulus of Elasticity (10⁶ psi)
16F-1.3E	1600	925	315	195	1.3
20F-1.5E	2000	1100	425	210	1.5 ⁵
24F-1.7E	2400	1450	500	210	1.7
24F-1.8E	2400	1450 ²	650	265 ⁴	1.8
26F-1.8E	2600	1950	650	265 ⁴	1.9
28F-1.8E	2800	2300	740	300	2.1 ⁶
30F-2.1E SP ⁷	3000	2400	740	300	2.1 ⁶
30F-2.1E LVL ⁸	3000	3000	510 ⁹	300	2.1

Notes:

¹For balanced layups, bending stress when compression zone is stressed in tension shall be equal to the bending stress when tension zone is stressed in tension for the stress class. Designer shall specify when balanced layup is required.

²Negative bending stress, is permitted to be increased to 1850 psi for Douglas-fir and to 1950 psi for southern pine for specific combinations. Designer shall specify when these increased stresses are required.

³For non-prismatic members, notched members, and members subjected to impact or cyclic loading, the design value for shear shall be multiplied by a factor of 0.72.

⁴Shear Parallel to Grain (Horizontal) = 300 psi for glulam made of southern pine.

⁵Modulus of Elasticity may be increased to 1.8×10^6 psi for glulam made of Canadian spruce-fir-pine or Eastern spruce.

⁶Modulus of Elasticity = 2.0×10^6 psi for members with more than 15 laminations except for 30F hybrid LVL glulam.

⁷Limited to a maximum of 6 inches except for hybrid LVL glulam.

⁸Requires the use of an outermost LVL lamination on the top and bottom.

⁹Compressive perpendicular to grain stress can be increased to the published value for the outer most LVL lamination.

TABLE 10.12 Design Value Properties of Select Glulams Produced by APA-EWS Members (APA-EWS, 2004).

Bending (Loaded perpendicular to wide faces of laminations)										
Combination Symbol	Species Outer/Core	Balanced/Unbalanced	Extreme Fiber in Bending				Compression Perpendicular to Grain		Shear Parallel to Grain	Modulus of Elasticity
			Tension Zone Stressed in Tension	Compression Zone Stressed in Tension	Tension Face	Compression Face	Tension Face	Compression Face		
Western Species										
EWS 20F-E/ES1	ES/ES	B	2000	2000	2000	560	560	200	1.8	
EWS 24F-E/ES1	ES/ES	U	2400	1700	560	560	560	200	1.7	
EWS 24F-V4	DF/DF	U	2400	1850	650	650	650	265	1.8	
EWS 24F-V8	DF/DF	B	2400	2400	650	650	650	265	1.8	
Southern Pine										
EWS 24F-V3	SP/SP	U	2400	1950	740	740	740	300	1.8	
EWS 24F-V5	SP/SP	B	2400	2400	740	740	740	300	1.7	

TABLE 10.12 (Continued)

		Bending (Loaded perpendicular to wide faces of laminations)				Axial Loaded	
Combination Symbol	(psi)	Compression		(10 ⁶ psi)	(psi)	(psi)	(10 ⁶ psi)
		Extreme Fiber in Bending	Perpendicular to Grain				
		(psi)	(psi)	(10 ⁶ psi)	(psi)	(psi)	(10 ⁶ psi)
Western Species							
EWS 20F-E/ES1	1100	300	175	1.5	1050	1150	1.6
EWS 24F-E/ES1	1100	300	175	1.5	1050	1150	1.6
EWS 24F-V4	1450	560	230	1.6	1100	1650	1.7
EWS 24F-V8	1450	560	230	1.6	1100	1650	1.7
Southern Pine							
EWS 24F-V3	1750	650	265	1.6	1150	1650	1.7
EWS 24F-V5	1750	650	265	1.5	1150	1650	1.6

or whether the wood in the laminate is visually graded or stress graded. Species may be specified in combination with these stress classes to obtain the required design properties. The presence of a stress classification does not ensure that the material is available. In addition, higher design values for some properties may be obtained by specifying a specific combination of laminations. Table 10.12 presents a partial list of allowable design stresses for glulams, with four or more laminations, used as bending members that are commonly produced by members of the APA-EWS. The species of wood in this table are

ES: Eastern spruce

DF: Douglas fir

SP: Southern pine

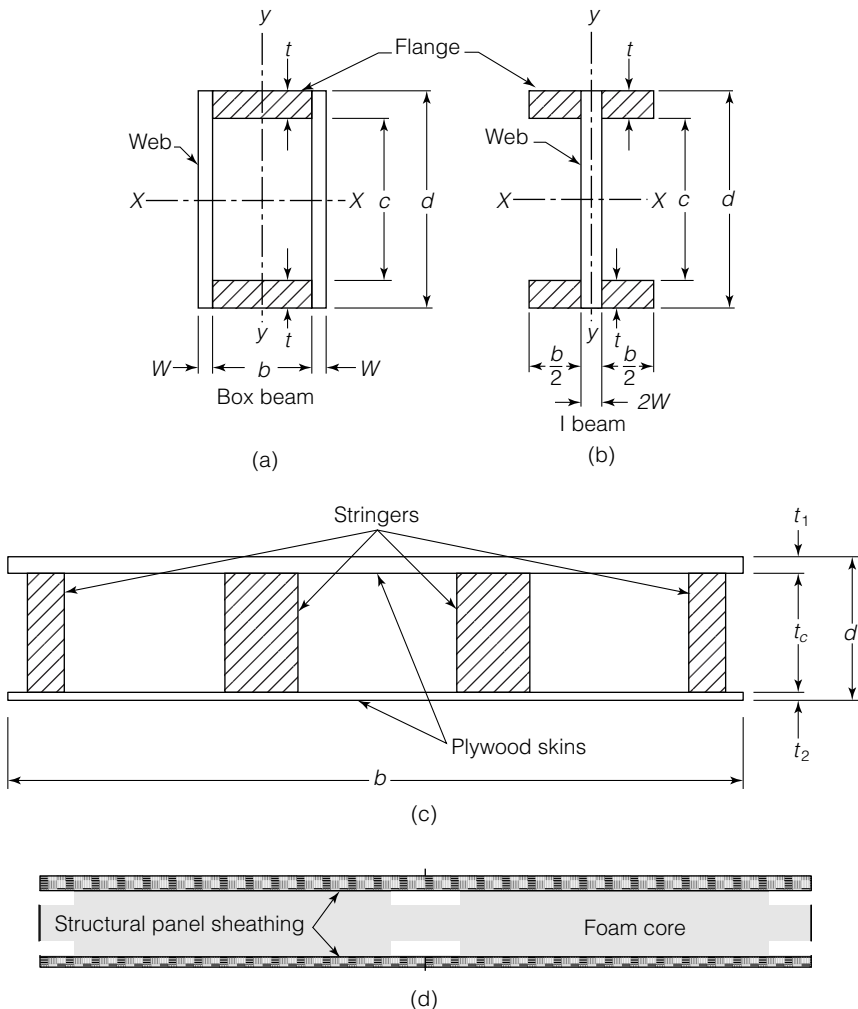


FIGURE 10.21 Composite structural members (a) box beam, (b) I-beam, (c) stressed skin panel, and (d) structural insulated panel.

It should be noted that there are many footnotes associated with this table, and before use of this information, the reader should consult the cited reference (APA-EWS, 2004). Table 10.12 addresses design values only for bending members. A similar table is available in the cited reference for design values for members subjected to axial tension or compression.

10.13.3 ■ Composite Structural Members

As with natural wood products, glued products can be combined in a variety of ways to produce structural elements. Typical examples of manufactured composite members (Figure 10.21) are

- I-beams
- Box beams
- Stressed skinned panels
- Structural insulated panels

For example, I-beams are produced by gluing flange elements to an oriented strand board web, as shown in Figure 10.21(b). The flange elements may be either dimensional lumber or, more commonly, laminated veneer lumber.

As manufactured products, the properties of the elements are frequently specified based on the performance of the member, rather than being based on the individual material components. For example, I-joists are specified based on the spacing of the joists and the depth of the member. Standards for composite structural members may be developed by national organizations, such as ASTM, or by trade associations concerned with the proper use of the products they promote.

S U M M A R Y

Wood is an extremely flexible building material. Historically, natural wood products were the only option available to the engineer. However, modern forestry practices limit the size of natural products that are available. The need to increase the efficiency of using wood products has led to the development of engineered wood products. These products are frequently more economical than natural wood, particularly when large dimensions are required. In addition, by careful control of the manufacturing process, engineered woods can be produced that have characteristics superior to natural wood. The characteristics of engineered wood products depend on the wood stock used, the quality of the adhesive, and the manufacturing process. Several factors make wood unique when compared to the other materials used in civil engineering, including anisotropy, moisture sensitivity, creep, and the existence of defects in wood products. Furthermore, when wood is exposed to the environment, care must be taken to prevent degradation due to fungi, bacteria, and insects.

Q U E S T I O N S A N D P R O B L E M S

- 10.1 What are the two main classes of wood? What is the main use of each class? State the names of two tree species of each class.
- 10.2 What is the difference between early wood and latewood? Describe each.
- 10.3 A simple lab test for specific gravity on two samples of lumber indicate that sample A has $G = 1.12$ and sample B has $G = 1.03$. Based on this information alone, which wood sample would you choose as a structural member for your construction project. Briefly explain why.
- 10.4 Discuss the anisotropic nature of wood. How does this phenomenon affect the performance of wood?
- 10.5 Briefly describe the chemical composition of wood.
- 10.6 The moisture content of wood test was performed according to ASTM D4442 procedure and produced the following data:
Weight of specimen in the green condition = 266.7 g.
Weight of oven-dry specimen = 152.1 g.
Calculate the moisture content of the given wood.
- 10.7 What is the fiber saturation point? What is the effect of the fiber saturation point on the shrinkage of wood in the different directions? How does this phenomenon affect the properties of lumber?
- 10.8 A stud had dimensions of 38.1 mm \times 88.9 mm \times 2.438 m and a moisture content of 150% when it was prepared. After seasoning, the moisture content was reduced to 7%. If the tangential, radial, and longitudinal directions of the grains are on the same order as the dimensions indicated above, what are the dimensions of the seasoned stud if the moisture-shrinkage relation follows Figure 10.5?
- 10.9 A wood timber with a diameter of 1 inch has a moisture content of 5%. The fiber saturation point (FSP) for this wood is 28%. The wood shrinks or swells 1% in the radial direction for every 5% change in moisture content below FSP.
- What would be the percent change in the wood's diameter if the wood's moisture is increased to 43%?
 - Would the wood swell or shrink?
 - What would be the new diameter?
- 10.10 Wood is cut at sawmills into a variety of products, with different sizes and shapes for engineering applications. What are these products?
- 10.11 Construction lumber can be cut from the tree using one of two methods or a combination of them. Name these two methods and show a sketch of each. What is the main advantage of each method?

- 10.12 Why are the actual dimensions of lumber different from the nominal dimensions? Explain.
- 10.13 What are the factors considered in grading lumber? What are the main grades of hardwoods and softwoods?
- 10.14 State five different imperfections that may be found in lumber, and briefly define them.
- 10.15 Sketch the typical stress–strain curve for wood. On the graph, show the modulus of elasticity.
- 10.16 Compute the modulus of elasticity of the wood species whose stress–strain relation is shown in Figure 10.9, using both the SI and English units. Compare the results with the typical values shown in Table 1.1 and comment about the results.
- 10.17 List five different tests used to evaluate the mechanical properties of wood.
- 10.18 A wood specimen was subjected to bending until failure by applying a load in the middle of its span. The specimen has a cross section of 1 in. \times 1 in. and a span of 14 in. The load and the deflection in the middle of the span were recorded as shown in Table P10.18.
- Using a computer spreadsheet program, plot the load–deflection relationship.
 - Plot the proportional limit on the graph.
 - Calculate the modulus of rupture (flexure strength).

Table P10.18

Load (lb)	Deflection (in.)
0	0
8	0.149
16	0.227
28	0.295
60	0.365
111	0.404
156	0.437
198	0.468
243	0.502
286	0.537
328	0.581
365	0.616

- 10.19. A wood specimen having a square cross section of 2 in. \times 2 in. was tested in bending by applying a load at the middle of the span, where the span was 28 in. The deflection under the load was measured at different load levels as shown in Table P10.19.

Table P10.19

Load (lb)	Deflection (10^{-3} in.)
0	0
100	27.9
200	55.6
300	83.2
400	111.2
500	140.0
600	166.7
700	194.3
800	222.2
900	250.1
1000	275.4
1100	314.8
1200	359.5
1300	405.0
1400	468.6 (failure)

- a. Using a computer spreadsheet program plot the load–deflection relationship.
 - b. Plot the proportional limit on the graph.
 - c. Calculate the modulus of rupture (flexure strength).
 - d. Does the modulus of rupture computed in (c) truly represent the extreme fiber stresses in the specimen? Comment on the assumptions used to compute the modulus of rupture and the actual response of the wood specimen.
- 10.20 A wood specimen was prepared with actual dimensions of 1 in. \times 1in. \times 4 in. and grains parallel to its length. The specimen was subjected to compression parallel to the grains to failure. The load–deformation results are as shown in Table P10.20.
- a. Using a computer spreadsheet program, plot the stress–strain relationship.
 - b. Calculate the modulus of elasticity.
 - c. What is the failure stress?
- 10.21 A pine wood specimen was prepared with dimensions of 50 mm \times 50 mm \times 200 mm and grains parallel to its length. The specimen was subjected to compression parallel to the grains to failure. The load–deformation results are as shown in Table P10.21.
- a. Using a computer spreadsheet program, plot the stress–strain relationship.
 - b. Calculate the modulus of elasticity.
 - c. What is the failure stress?

Table P10.20

Load (lb)	Displacement (in.)
0	0
7	0.012
10	0.068
87	0.164
530	0.180
1705	0.208
2864	0.236
3790	0.268
4606	0.300
5338	0.324
5116	0.360
4468	0.384
4331	0.413

Table P10.21

Load (kN)	Deformation (mm)
0	0
8.9	0.457
17.8	0.597
26.7	0.724
35.6	0.838
44.5	0.965
53.4	1.118
62.3	1.270
71.2	1.422
80.1	1.588
89.0	1.765
97.9	1.956
106.8	2.159
111.3	2.311

- 10.22 A wood specimen was prepared with dimensions of 1 in. \times 1 in. \times 4 in. and grains parallel to its length. The specimen was subjected to compression parallel to the grains to failure. The load (P) versus deformation (ΔL) results are as shown in Table P10.22. Using a spreadsheet program, complete the table by calculating engineering

Table P10.22

Observation No.	<i>P</i> (lb)	ΔL (in.)	σ (psi)	ε (in./in.)	u_i (psi)
0	0	0			N/A
1	720	0.020			
2	1720	0.048			
3	2750	0.076			
4	3790	0.108			
5	4606	0.140			
6	5338	0.164			
7	6170	0.200			
8	6480	0.224			
9	5400	0.253			
					$u_t =$

stress (σ) and engineering strain (ε). Determine the toughness of the material (u_t) by calculating the area under the stress–strain curve, namely,

$$u_t = \int_0^{\varepsilon_f} \sigma \, d\varepsilon$$

where ε_f is the strain at fracture. This integral can be approximated numerically using a trapezoidal integration technique:

$$u_t = \sum_{i=1}^n u_i = \sum_{i=1}^n \frac{1}{2} (\sigma_i + \sigma_{i-1}) (\varepsilon_i - \varepsilon_{i-1})$$

- 10.23 A short rounded wood column with a diameter of 10 inches is to be constructed. If the failure stress is 7.3 ksi, what is the maximum load that can be applied to this column, using a factor of safety of 1.3?
- 10.24 For the purpose of designing wood structures, laboratory-measured strength properties are adjusted for application conditions. State five different application conditions that are used to adjust the strength properties.
- 10.25 What are the five types of organisms that attack wood?
- 10.26 What are the two types of preservatives that can be used to protect wood from decay? How are these preservatives applied?
- 10.27 What are the main types of engineered wood products?
- 10.28 What are the main advantages of engineered wood products over natural-timber members?

10.14 References

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11

COMPOSITES



The need for materials with properties not found in conventional materials, combined with advances in technology, have resulted in combining two or more materials to form what are called composite materials. These materials usually combine the best properties of their constituents and frequently exhibit qualities that do not even exist in their constituents. Strength, stiffness, specific weight, fracture resistance, corrosion resistance, wear resistance, attractiveness, fatigue life, temperature susceptibility, thermal insulation, thermal conductivity, and acoustical insulation can all be improved by composite materials. Of course, not all these properties are improved in the same composite, but typically a few of these properties are improved. For example, materials needed to build aircraft and space vehicles must be light, strong, and stiff and must exhibit high resistance to abrasion, impact, and corrosion. An example of a composite material that is very useful for civil engineers is fiberglass, which is strong, stiff, and corrosion resistant and can be used to make concrete reinforcing rebars to replace corrosive steel rebars. These combinations of properties are formidable and typically cannot be found in a conventional material.

Composite materials have been used throughout history, with differing levels of sophistication. For example, straw was used to strengthen the mud bricks in ancient civilizations. Swords and armor were constructed with layers of different materials to obtain unique properties. Portland cement concrete, which combines paste and aggregate with different properties to form a strong and durable construction material, has been used for many years. In recent years, fiber-reinforced concrete has been used as a building material that is strong in both tension and compression. The automobile industry has been using composite metals to build lightweight vehicles that are strong and impact resistant. Recently, a new generation of composites has been developed, such as fiber-reinforced and particle-reinforced plastics, that has revolutionized the material industry and opened new horizons for civil and construction engineering applications.

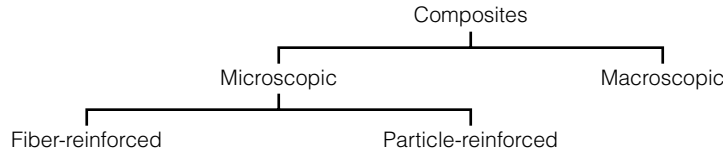


FIGURE 11.1 A classification scheme of composite materials.

Although several definitions of composites exist, it is generally accepted that a composite is a material that has two or more distinct constituent materials or phases. The constituents of a composite typically have significantly different physical properties, and thus the properties of the composite are noticeably different from those of the constituents. This definition eliminates many multiphase materials that do not have distinct properties, such as many alloys with components that are similar.

There are a number of naturally formed composites, such as wood, which consists of cellulose fibers and lignin, and bone, which consists of protein collagen and mineral appetite. However, in this chapter we discuss artificially made composites only.

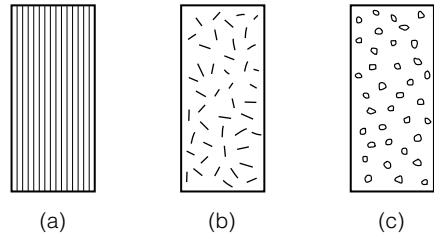
Composite materials can be classified as *microscopic* or *macroscopic*, as shown in Figure 11.1. The distinction between microscopic and macroscopic depends on the type of properties being considered. This distinction seems arbitrary, but normally microscopic composites include *fibers* or *particles* in sizes up to a few hundred microns. On the other hand, macroscopic composites could have constituents of much larger size, such as aggregate particles and rebars in concrete.

11.1 Microscopic Composites

Many microscopic composite materials consist of two constituent phases: a continuous phase, or *matrix*, and the *dispersed phase* or *reinforcing phase*, which is surrounded by the matrix. In most cases, the dispersed phase is harder and stiffer than the matrix. The properties of the composite depend on the properties of both component phases, their relative properties, and the geometry of the dispersed phase, such as the particle shape, size, distribution, and orientation.

As indicated in Figure 11.1, microscopic composites fall into two basic classes: *fiber-reinforced* and *particle-reinforced*. This classification is based on the shape of the dispersed phase. Figure 11.2 shows composites with continuously aligned fibers, random fibers, and random particles. The mechanism of strengthening varies for different classes and for different sizes and orientations of the dispersed shape.

FIGURE 11.2 Schematic of microscopic composites: (a) aligned fibers, (b) random fibers, and (c) random particles.



11.1.1 ■ Fiber-Reinforced Composites

Fiber-reinforced microscopic composites include fibers dispersed in a matrix such as metal or polymer. Fibers have a very high strength-to-diameter ratio, with near crystal-sized diameters. Because of the very small diameter of the fibers they are much stronger than the bulk material. For example, a glass plate fractures at stresses of 10 kPa to 20 kPa, yet glass fibers have strengths of 3 MPa to 5 MPa or more. Fibers are much stronger than the bulk form, because they have fewer internal defects.

Fibers can be classified on the basis of their diameter and character as whiskers, fibers, and wires. Whiskers are very thin single crystals, have extremely large length-to-diameter ratios, have a high degree of crystalline perfection, and, consequently are extremely strong. Fibers have larger diameters than whiskers, while wires are even larger. Whiskers are not commonly used for reinforcement, because of their high cost, poor bond with many common matrix materials, and the difficulty of incorporating them into the matrix. Table 11.1 shows some materials used to manufacture fibers and their strength characteristics.

Fibers are manufactured from many materials, such as glass, carbon and graphite, polymer, boron, ceramic, and silicon carbide (Mallick 1993). Because of their low cost and high strength, glass fibers are the most common of all reinforcing fibers for polymer matrix composites. Glass fibers are

TABLE 11.1 Materials and Mechanical Properties of Some Fibers (Callister, 2003)

Material	Specific Gravity	Tensile Strength, GPa (psi $\times 10^6$)	Elastic Modulus, GPa (psi $\times 10^6$)
Aramid (Kevlar)	1.4	3.5 (0.5)	130 (19)
E-Glass	2.5	3.5 (0.5)	72 (10.5)
Graphite	1.4	1.7 (0.25)	255 (37)
Nylon 6,6	1.1	1.0 (0.14)	4.8 (0.7)
Asbestos	2.5	1.4 (0.2)	172 (25)



FIGURE 11.3 Common glass fibers (veil, roving, and mat). Photo courtesy of Creative Pultrusions, Inc.

commercially available in several forms suitable for different applications. Common glass fibers include veils, rovings (continuous fibers), and mats (Figure 11.3). A strand consists of about 30 or 40 fibers twisted together to form a ropelike length.

A common fiber-reinforced composite is fiberglass. Fiberglass is simply a composite consisting of glass fibers, either continuous or discontinuous, contained within a plastic matrix. Typical fiberglass applications include aircraft, automobiles, boats, storage containers, water tanks, sporting equipment, and flooring.

11.1.2 ■ Particle-Reinforced Composites

Particle-reinforced composites consist of particles dispersed in a matrix phase. The strengthening mechanism of particle-reinforced composites varies with the size of the reinforcing particles. When the size of the particles is about 0.01 micron to 0.1 micron, the matrix bears most of the applied load, whereas the small dispersed particles hinder or impede the motion of dislocations. An example of dispersed-reinforced composite is thoria-dispersed nickel, in which about 3% of thoria (ThO_2) is finely dispersed in a nickel alloy to increase its high-temperature strength. On the other hand, when the particles are larger than 1 micron, particles act as fillers to improve the properties of the matrix phase and/or to replace some of its volume, since the filler is typically less expensive. Here, the matrix retains movement in the vicinity of the particle. Thus, the applied load is shared by the matrix and dispersed phases. The stronger the bond between the dispersed particles and the matrix, the larger is the reinforcing effect. An example of particle reinforcing is adding fillers to polymers to improve tensile and compressive strengths, abrasion resistance, toughness, dimensional and thermal properties, and other properties.

11.1.3 Matrix Phase

Typically, the matrix used in most microscopic composites is polymer (plastic) or metal. The matrix binds the dispersed materials (particles or fibers) together, transfers loads to them, and protects them against environmental attack and damage due to handling. Polymers have the advantages of low cost, easy processibility, good chemical resistance, and low specific gravity. The shortcomings of polymers are their low strength, low modulus, low operating temperatures, and low resistance to prolonged exposure to ultraviolet light and some solvents. On the other hand, metals have high strength, high modulus, high toughness and impact resistance, relative insensitivity to temperature changes, and high resistance to high temperatures and other severe environmental conditions. However, metals have high density and high processing temperatures, due to their high melting points. Metals also may react with particles and fibers and they are vulnerable to attack by corrosion (Agarwal and Broutman 1990). The metals most commonly used as the matrix phase in composites are aluminum and titanium alloys.

11.1.4 Fabrication

Fabrication of microscopic composites often combines the production of the material during the fabrication of the composite. The composite is formed by combining the matrix and dispersed material. Several methods have been used to fabricate the composites. The selection of the fabrication process typically is based on the chemical nature of the matrix and of the dispersed phases and on the temperature required to form, melt, or cure the matrix. Figure 11.4 illustrates fabrication of structural shape fiber-reinforced composites by using the pultrusion process. Pultrusion is an automated process for manufacturing fiber-reinforced composite materials into continuous, constant-cross-section profiles.

11.1.5 Civil Engineering Applications

Microscopic composites have been used in many civil and construction engineering applications in the last several decades. In fact, composite materials

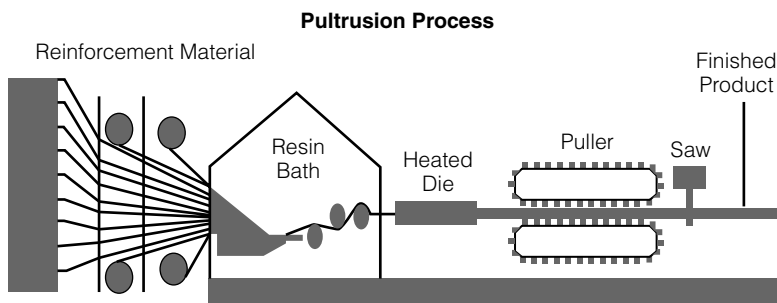


FIGURE 11.4 Pultrusion scheme used in fabricating structural shape fiber-reinforced composites. Courtesy of Creative Pultrusions, Inc.

compete with, and in many cases are preferred over, conventional building materials. Composites are used by civil engineers as structural shapes in buildings and other structures and can replace steel and aluminum structural shapes (Figure 11.5). Table 11.2 provides an example of physical properties of fiber-reinforced composite round rods and bars.

Fiber-reinforced polymer (fiberglass) rebars can also be used for concrete reinforcement instead of steel rebars. Composites have been used for tanks, industrial flooring, trusses and joists, walkways and platforms, waste treatment plants, handrailings, plastic pipes, light poles, door and window panels and frames, and electrical enclosures. Composites can also be used to strengthen and wrap columns and bridge supports that are partially damaged by earthquakes and other environmental factors (Grace 2002) (See Figures 11.6 – 11.9).

Fiber-reinforced concrete is another composite material that has been used by civil engineers in various structural applications. Different types of fibers, such as separate fibers, chopped-strands, or rovings, can be used to reinforce the concrete. If separate fibers or chopped-strands are used, they are mixed with the fresh concrete in a random order. In such a case, fibers hinder or impede the progression of cracks in concrete. Figure 11.10 shows a scanning electron micrograph of concrete mortar mixed with about 3-mm-long carbon fibers at a volume fraction of 12%. Fiber rovings, on the other hand, are placed in the direction in which the tension is applied in the structural member. In this case, fibers carry the tensile stresses. In general, fibers increase the tensile and flexure strength of concrete so that a more efficient structural member can be designed. Table 11.3 shows typical ranges of physical properties of glass fiber-reinforced concrete at 28 days. Research has shown that glass fiber-reinforced concrete offers two to three times the flexural strength of unreinforced concrete. Moreover, the material under increasing load does not fail abruptly, but yields gradually. This gradual yielding occurs because

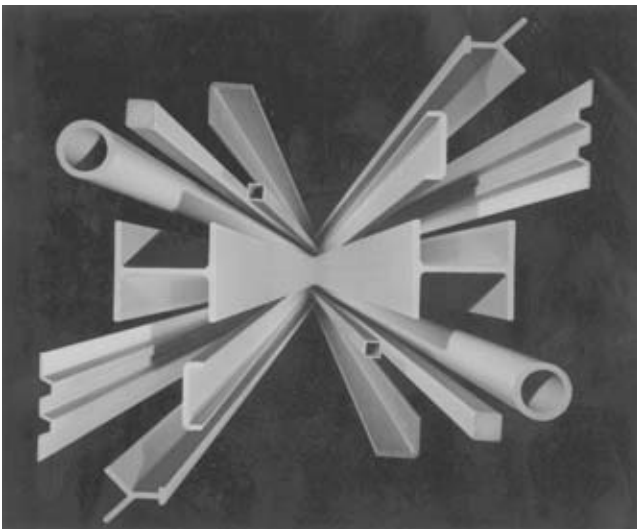


FIGURE 11.5 Structural shapes made of fiberglass composites. Courtesy of Creative Pultrusions, Inc.

TABLE 11.2 An Example of Physical Properties of Fiber-Reinforced Composite Round Rods and Bars (Creative Pultrusions, Inc., 1997)

Property	Value
Tensile strength (ASTM D638)	830 MPa (120×10^3 psi)
Tensile modulus of elasticity (ASTM D638)	45 GPa (6.5×10^6 psi)
Flexural strength (ASTM D790)	830 MPa (120×10^3 psi)
Compressive strength (ASTM D695)	480 MPa (70×10^3 psi)
Izod impact strength (ASTM D256)	2.1 kJ/m (40 ft-lb/in.)
Barcol hardness (ASTM D2583)	50
Water absorption (ASTM D570)	0.25% (maximum)
Specific gravity (ASTM D792)	2.0
Coefficient of thermal expansion (ASTM D696)	5.2×10^{-6} m/m/°C (9.4×10^{-6} in./in./°F)



FIGURE 11.6 Wrapping an old structure with glass fiber-reinforced polymer (FRP) composite (Wabash Avenue pier, Baltimore, MD).



FIGURE 11.7 Wrapping columns of a bridge with FRP composites.

fibers are stronger than the matrix and, therefore, arrests cracks. Therefore, instead of a worsening of the first crack that occurs in the concrete, more cracks are developed elsewhere, and failure finally occurs when fibers pull out or break (Neal 1977).

Entrained air in concrete can also be considered as a component in a microscopic composite material. Entrained air increases the durability of concrete since it releases internal stresses due to freezing of water within the concrete. For the same water-to-cement ratio, however, air bubbles reduce the concrete strength by about 20%. Since entrained air also improves the workability of fresh concrete, the water-to-cement ratio can be reduced to compensate for some of the strength reduction.

11.2 Macroscopic Composites

Macroscopic composites are used in many engineering applications. Because macroscopic composites are relatively large, how the load is carried and how the properties of the composite components are improved vary from one composite to another. Common macroscopic composites used by civil and construction engineers include plain portland cement concrete, steel-reinforced concrete, asphalt concrete, and engineered wood such as glued-laminated timber, and structural strand board.



(a)



(b)

FIGURE 11.8 Woodland viaduct over metro north railroad in Westchester County, NY: (a) reinforcing with FRP composite and (b) finished structure.



FIGURE 11.9 Strengthening a concrete bridge with FRP bars.

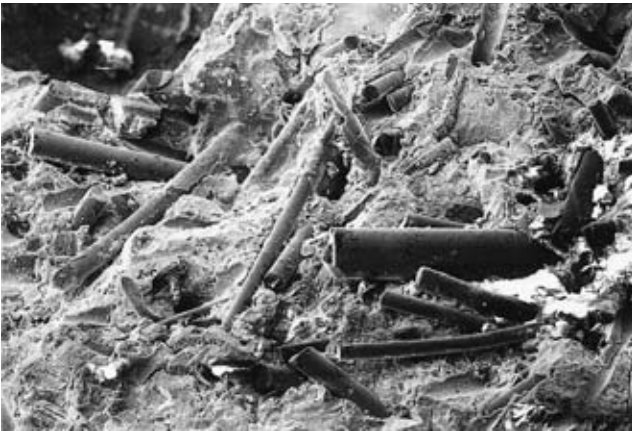


FIGURE 11.10 Scanning electron micrograph of concrete mortar mixed with carbon fibers.

TABLE 11.3 Typical Ranges of Physical Properties at 28 Days of Glass Fiber Reinforced Concrete (Neal, 1977)

Property	Value
Flexural strength	21–32 MPa (3.0–4.6 ksi)
Tensile strength	7–11 MPa (1.0–1.6 ksi)
Compressive strength	50–79 MPa (7.2–11.4 ksi)
Impact strength	10–25 kN/m (57–143 in. lb/in. ²)
Elastic modulus	10.5–20.5 GPa (1.5–3.0 × 10 ⁶ psi)
Density	1.70–2.10 Mg/m ³ (105–130 lb/ft ³)

11.2.1 ■ Plain Portland Cement Concrete

Plain portland cement concrete is a composite material consisting of cement paste and aggregate particles with different physical and mechanical properties, as discussed in Chapter 7 (Figure 11.11). Aggregate particles in concrete act as a filler material, since it is cheaper than the portland cement. In addition, since cement paste shrinks as it cures, aggregate increases the volume stability of the concrete. When the concrete structure is loaded, both cement paste and aggregate share the load. Both the strength of aggregate particles and the bond between the aggregate and cement paste play an important role in determining the strength of the concrete composite, which is limited by the weaker of the two. The bond between cement paste and aggregate is affected by roughness and absorption of the aggregate particles, as well as by other physical and chemical properties of aggregate.

11.2.2 ■ Reinforced Portland Cement Concrete

Steel-reinforced concrete can be viewed as a composite material, consisting of plain concrete and steel rebars, as shown in Figure 11.12. Since concrete has a very low tensile strength, which is typically ignored in designing concrete structures, steel rebars are usually placed in areas within the structure



FIGURE 11.11 Cross section of portland cement concrete showing cement paste and aggregate particles.

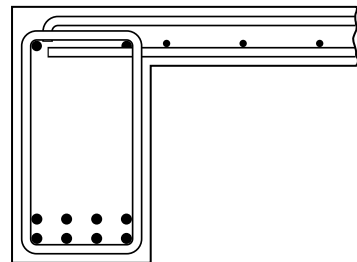


FIGURE 11.12 An example of steel-reinforced concrete beam and slab details.

that are subjected to tension. When the concrete structure is loaded, the concrete carries compressive stresses and steel carries tensile stresses. In such cases steel allows concrete to be used as structural members carrying tension. Steel rebars are also used in areas subjected to compression, such as columns, to share the load support. In such cases, steel reduces the required cross-sectional area of the compressive member, since the compressive strength of steel is larger than that of concrete. Steel reinforcing is also used in prestressed concrete, where the reinforcement is prestressed under tension so that the concrete remains under compression even when it is externally loaded. In such cases, a smaller cross-sectional area of the concrete member is required. Steel rebars can also be used to control cracking in concrete due to temperature change. For example, concrete pavement is sometimes reinforced by placing longitudinal and transverse steel bars at the midheight of the concrete slab. In this case, when the concrete shrinks due to reduction in ambient temperature, many cracks will develop; these cracks are uniformly distributed within the pavement section, but each crack will be tight. Typically, tight cracks are not harmful to concrete pavement, since they transfer the load from one side of the crack to the other by interlocking. In all applications of steel reinforced concrete, the bond between the rebars and the concrete is important in order to allow the composite to work as one unit. Therefore, bars have a deformed surface to prevent slipping between steel and concrete.

11.2.3 ■ Asphalt Concrete

Asphalt concrete used in pavements is another composite material. It consists of two materials with distinct properties, as presented in Chapter 9. Asphalt concrete consists of approximately 95% aggregate and 5% asphalt binder, by weight. When the traffic loads are applied on the asphalt concrete composite, most of the compressive stresses are supported by the aggregate-to-aggregate contact. The asphalt acts as a binder that prevents slipping of aggregate particles relative to each other. When tensile stresses are applied due to bending of the asphalt concrete layer or due to thermal contraction, the aggregate particles are supported by the asphalt binder. One important property of asphalt is that it gets soft at high temperatures and brittle at low temperatures, whereas aggregate does not change its properties with temperature fluctuation. It is important, therefore, to properly select the asphalt grade that will perform properly within the temperature range of the region in which it is being used. Also, since aggregate represents a major portion of the mixture, it is important to use aggregate with proper gradation and other properties. The asphalt binder content must be carefully designed in order to ensure that aggregate particles are fully coated, without excessive lubrication. When the asphalt concrete mixture is appropriately designed and compacted, it should last for a long time without failure.

11.2.4 ■ Engineered Wood

Engineered wood is manufactured by bonding together wood strands, veneers, or lumber with different grain orientations to produce large and integral units. Since engineered wood consists of components of the same material, it does not qualify as a composite according to our definition. However, engineered wood is presented in this chapter because it follows a strengthening mechanism similar to that of composites. Since wood has anisotropic properties due to the existence of grains, engineered wood produces specific and consistent mechanical behavior and thus has consistent design properties. For example, alternating the grain orientation of the plies of plywood provides nearly identical properties along the length and width and provides resistance to dimensional change under varying moisture conditions. The plywood composite has about one-tenth of the dimensional change of solid lumber under any temperature or moisture condition. As discussed in Chapter 10, engineered wood products include plywood, oriented strand boards, composite panels, glued–laminated timber (glulam), laminated veneer lumber, parallel strand lumber, oriented strand lumber, and wood I-joists.

11.3 Properties of Composites

The properties of composite materials are affected by the component properties, volume fractions of components, type and orientation of the dispersed phase, and the bond between the dispersed phase and the matrix. The properties of the composite can be viewed as the weighted average of the properties of the components (Shackelford 1996). Equations can be derived to estimate the composite properties under certain idealized material properties, loading patterns, and geometrical conditions. Assumptions that can be used to simplify the analysis include the following:

- Each component has linear, elastic, and isotropic properties.
- A perfect bond exists between the dispersed and matrix phases without slipping.
- The composite geometry is idealized and the loading pattern is parallel or perpendicular to reinforcing fibers.

11.3.1 ■ Loading Parallel to Fibers

When load is applied to an aligned fiber-reinforced composite parallel to the fibers, as seen in Figure 11.13(a), both matrix and fiber phases will deform equally. Thus, the strains of both phases will be the same (known as an *isostrain condition*) and are given by

$$\varepsilon_c = \varepsilon_m = \varepsilon_f = \varepsilon \quad (11.1)$$

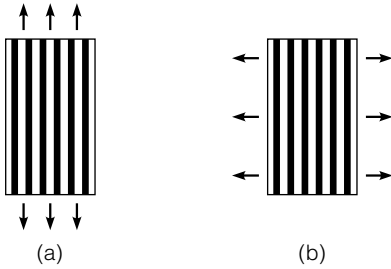


FIGURE 11.13 Patterns of loading continuously aligned fiber-reinforced composites: (a) loading parallel to fibers and (b) loading perpendicular to fibers.

where

- ε = total strain
- ε_c = composite
- ε_m = matrix strain
- ε_f = fiber strain

Also, the force applied to the composite F_c is the sum of the force carried by the matrix F_m and the force carried by the fibers F_f :

$$F_c = F_m + F_f \tag{11.2}$$

Thus,

$$\sigma_c A_c = \sigma_m A_m + \sigma_f A_f \tag{11.3}$$

where

- σ_i = stress of component i
- A_i = area of component i

Replacing σ with $E\varepsilon$ for each material, we can write Equation 11.3 as

$$E_c \varepsilon A_c = E_m \varepsilon A_m + E_f \varepsilon A_f \tag{11.4}$$

where E is the modulus of elasticity. Canceling ε and dividing by A_c , we have

$$E_c = E_m \frac{A_m}{A_c} + E_f \frac{A_f}{A_c} \tag{11.5}$$

or

$$E_c = \nu_m E_m + \nu_f E_f \tag{11.6}$$

where ν is the volume fraction of each component and $\nu_m + \nu_f = 1$.

Equation 11.6 shows that the composite's modulus of elasticity is the weighted average of the component moduli.

The share of the load carried by the fibers can be determined as follows:

$$\frac{F_f}{F_c} = \frac{\sigma_f A_f}{\sigma_c A_c} = \frac{E_f \varepsilon A_f}{E_c \varepsilon A_c} = \frac{E_f}{E_c} \nu_f \tag{11.7}$$

Sample Problem 11.1

Calculate the modulus of elasticity of fiberglass under isostrain condition if the fiberglass consists of 70% E-glass fibers and 30% epoxy by volume. Also, calculate the percentage of load carried by the glass fibers. The modulus of elasticity of the glass fibers and the epoxy are 70.5 GPa and 6.9 GPa, respectively.

Solution

From Equation 11.6,

$$E_c = (0.3)(6.9) + (0.7)(70.5) = 51.4 \text{ GPa}$$

From Equation 11.7,

$$\frac{F_f}{F_c} = \frac{70.5}{51.4}(0.7) = 0.96 = 96\%$$

This example shows that, under the given conditions, 96% of the load is carried by the fibers.

Equation 11.6 can be generalized to cover other composite properties as a function of the properties of the components as

$$X_c = v_m X_m + v_f X_f \quad (11.8)$$

where X is a property such as Poisson's ratio, thermal conductivity, electrical conductivity, or diffusivity.

11.3.2 Loading Perpendicular to Fibers

When load is applied to an aligned fiber-reinforced composite perpendicular to the fibers [Figure 11.13(b)], both matrix and fiber phases will be subjected to the same stress (isostress condition). In other words,

$$\sigma_c = \sigma_m = \sigma_f = \sigma \quad (11.9)$$

The elongation of the composite in the direction of the applied stress is the sum of the elongations of the matrix and fibers:

$$\Delta L_c = \Delta L_m + \Delta L_f \quad (11.10)$$

Dividing Equation 11.10 by the composite length L_c in the stress direction gives

$$\frac{\Delta L_c}{L_c} = \frac{\Delta L_m}{L_c} + \frac{\Delta L_f}{L_c} \quad (11.11)$$

Assuming that the fibers are uniform in thickness, the cumulative length of each component in the direction of the stress is proportional to its volume fraction. Thus,

$$L_m = \nu_m L_c \quad (11.12)$$

and

$$L_f = \nu_f L_c \quad (11.13)$$

Substituting the values of L_c from Equations 11.12 and 11.13 in Equation 11.11 yields

$$\frac{\Delta L_c}{L_c} = \frac{\nu_m \Delta L_m}{L_m} + \frac{\nu_f \Delta L_f}{L_f} \quad (11.14)$$

Since $\varepsilon = \frac{\Delta L}{L}$, Equation 11.14 can be rewritten as

$$\varepsilon_c = \nu_m \varepsilon_m + \nu_f \varepsilon_f \quad (11.15)$$

Replacing ε with $\frac{\sigma}{E}$ gives

$$\frac{\sigma}{E_c} = \nu_m \frac{\sigma}{E_m} + \nu_f \frac{\sigma}{E_f} \quad (11.16)$$

or

$$\frac{1}{E_c} = \frac{\nu_m}{E_m} + \frac{\nu_f}{E_f} \quad (11.17)$$

Equation 11.17 can be rewritten as

$$E_c = \frac{E_m E_f}{\nu_m E_f + \nu_f E_m} \quad (11.18)$$

As with Equation 11.8, Equation 11.18 can be generalized as

$$X_c = \frac{X_m X_f}{\nu_m X_f + \nu_f X_m} \quad (11.19)$$

where X is a property such as thermal conductivity, electrical conductivity, or diffusivity.

The moduli in Equations 11.6 and 11.18 can be plotted as functions of the volume fraction of the fiber, as shown in Figure 11.14. Clearly, the fibers are more effective in raising the modulus of the composite when loading parallel to fibers than when loading perpendicular to fibers.

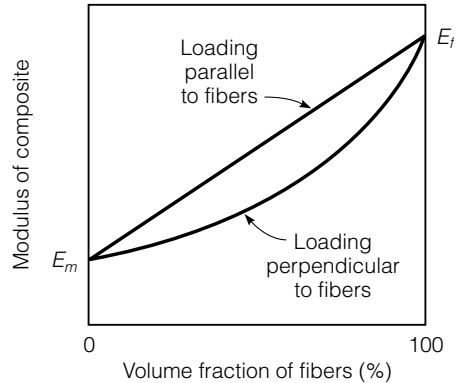


FIGURE 11.14 Modulus of elasticity of the composite versus fiber volume fraction.

11.3.3 ■ Randomly Oriented Fiber Composites

Unlike continuously aligned fiber composites, the mechanical properties of randomly oriented fiber composites are isotropic. The modulus of elasticity of randomly oriented fiber composites falls between the moduli of loading parallel to fibers and perpendicular to fibers. To estimate the modulus of elasticity of randomly oriented fiber composites, Equation 11.6 can be rewritten as

$$E_c = \nu_m E_m + K \nu_f E_f \quad (11.20)$$

where K is a fiber efficiency parameter (Callister 1985). For fibers randomly and uniformly distributed within three dimensions in space, K has a value of 0.2.

Sample Problem 11.2

A fiberglass composite consists of epoxy matrix reinforced with randomly oriented and uniformly distributed E-glass fibers. The modulus of elasticity of the glass fibers and the epoxy are 65 GPa and 7 GPa, respectively. If the volume percentage of fibers is 30%, and the fiber efficiency is 0.2, calculate the modulus of elasticity of the fiberglass.

Solution

From Equation 11.20, we have

$$E_c = 0.67 \times 7 + 0.2 \times 0.33 \times 65 = 9.0 \text{ GPa}$$

11.3.4 ■ Particle-Reinforced Composites

The analysis of loading a particle-reinforced composite depends on the specific nature of the dispersed and matrix phases. A rigorous analysis of loading a particle-reinforced composite can become quite complex. Equations 11.6 and 11.18 serve as upper and lower bounds for the particle-reinforced properties.

S U M M A R Y

Combining different materials to produce a composite that has properties superior to the component materials has been practiced since ancient times. In fact, many of the conventional materials currently used in civil engineering are composites, including portland cement concrete, reinforced concrete, asphalt concrete, and engineered woods. Composites are generally classified as either fiber or particle reinforced, depending on the nature of the dispersed phase material. The properties of composites depend on the characteristics of the component materials, the bonding between the dispersed and matrix phases, and the orientation of the dispersed phase.

Q U E S T I O N S A N D P R O B L E M S

- 11.1 What is a composite material? List some composite materials that you use in your daily life.
- 11.2 List five different advantages of composite materials over conventional materials.
- 11.3 Define microscopic composites. What are the two phases of microscopic composites?
- 11.4 What are the two types of microscopic composites? Show the mechanism for strengthening of each type.
- 11.5 Why are fibers much stronger than the bulk material? Give an example of a material that is relatively weak in the bulk form and very strong in the fiber form.
- 11.6 Compare the desired properties of the matrix and the fiber phases of the fiber-reinforced composite.
- 11.7 Name three functions of the matrix phase in fiber-reinforced composites. State the reason for the need for a strong bond between the fibers and the matrix.
- 11.8 What are the functions of aggregate used in portland cement concrete?
- 11.9 How is the load supported by asphalt concrete in the cases of tension and compression. Under what conditions is the asphalt concrete layer subjected to tension?

- 11.10 Briefly describe why engineered wood is stronger and has better properties than natural wood.
- 11.11 Calculate the modulus of elasticity of carbon–epoxy composite under isostrain condition if the composite consists of 30% carbon fibers and 70% epoxy by volume. Also, calculate the percentage of load carried by the carbon fibers. The modulus of elasticity of the carbon fibers and the epoxy are 50×10^6 psi and 0.5×10^6 psi, respectively.
- 11.12 Repeat problem 11.11 for 40% carbon fibers by volume.
- 11.13 Repeat problem 11.11 under isostress condition.
- 11.14 Calculate the modulus of elasticity of carbon–epoxy composite under isostrain condition if the composite consists of 50% carbon fibers and 50% epoxy by volume. Also, calculate the percentage of load carried by the carbon fibers. The modulus of elasticity of the carbon fibers and the epoxy are 350 GPa and 3.5 GPa, respectively.
- 11.15 Repeat problem 11.14 for 30% carbon fibers by volume.
- 11.16 Repeat problem 11.14 under isostress condition.
- 11.17 A fiberglass composite consists of epoxy matrix reinforced with randomly oriented and uniformly distributed E-glass fibers. The modulus of elasticity of the glass fibers and the epoxy are 70 GPa and 6 GPa, respectively. Calculate the modulus of elasticity of the fiberglass if the volume percentage of fibers is (a) 25%, (b) 50%, and (c) 75%. Plot a graph showing the relationship between the modulus of elasticity of the fiberglass and the percent of fibers. Comment on the effect of the percent of glass fibers on the modulus of elasticity of fiberglass.
- 11.18 A fiberglass composite consists of epoxy matrix reinforced with randomly oriented and uniformly distributed E-glass fibers. The modulus of elasticity of the glass fibers and the epoxy are 10×10^6 psi and 1×10^6 psi, respectively. Calculate the modulus of elasticity of the fiberglass if the volume percentage of fibers is (a) 30%, (b) 50%, and (c) 70%. Plot a graph showing the relationship between the modulus of elasticity of the fiberglass and the percent of fibers. Comment on the effect of the percent of glass fibers on the modulus of elasticity of fiberglass.
- 11.19 A short reinforced concrete column is subjected to a 1000 kN axial compressive load. The moduli of elasticity of plain concrete and steel are 25 GPa and 207 GPa, respectively, and the cross-sectional area of steel is 2% of that of the reinforced concrete. Considering the column as a structural member made of a composite material and subjected to load parallel to the steel rebars, calculate the following:
- the modulus of elasticity of the reinforced concrete
 - the load carried by each of the steel and plain concrete
 - the minimum required cross-sectional area of the column given that the allowable compressive stress of plain concrete is 20 MPa and that the allowable compressive stress of plain concrete will be reached before that of steel.

- 11.20 A short reinforced concrete column is subjected to a 500 kips axial compressive load. The moduli of elasticity of plain concrete and steel are 4.5×10^6 psi and 30×10^6 psi, respectively, and the cross-sectional area of steel is 1.8% of that of the reinforced concrete. Considering the column as a structural member made of a composite material and subjected to load parallel to the steel rebars, calculate the following:
- the modulus of elasticity of the reinforced concrete
 - the load carried by each of the steel and plain concrete
 - the minimum required cross-sectional area of the column, given that the allowable compressive stress of plain concrete is 5000 psi and that the allowable compressive stress of plain concrete will be reached before that of steel.

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