Civil Engineering Materials

Nagaratnam Sivakugan
James Cook University, Australia

Carthigesu T. Gnanendran
The University of New South Wales
at the Australian Defence Force
Academy, Australia

Rabin Tuladhar
James Cook University, Australia

M. Bobby Kannan
James Cook University, Australia

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Authors: Nagaratnam Sivakugan,
Carthigesu T. Gnanendran,
Rabin Tuladhar, and M. Bobby Kannan

Product Director, Global Engineering: Timothy L. Anderson

Associate Media Content Developer: Ashley Kaupert

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Marketing Manager: Kristin Stine

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Content Project Manager: Megan Guiliani

Production Service: RPK Editorial Services, Inc.

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PREFACE

Materials science and engineering is a multidisciplinary area that is offered in undergraduate and graduate programs at many leading universities. It covers all engineering materials such as metals, ceramics, plastics, composites, and nanomaterials. When it comes to traditional engineering undergraduate programs such as civil, mechanical, electrical, or chemical engineering, their specific materials science educational needs are quite different. While civil engineers deal mostly with steel, concrete, timber, and soils, their mechanical engineering counterparts are interested in different alloys and composite materials. With rapid economic development and the scarcity of natural resources, the use of synthetic materials (e.g., polymers, composites), industrial byproducts (e.g., slag, fly ash), recycled materials and their combinations with traditional materials (e.g., concrete and soils) has recently become more prevalent in civil engineering projects. Hence, there is a growing need for civil engineers to learn more about these advanced materials in addition to traditional materials.

Civil engineering mainly deals with the design and construction of civil infrastructure (e.g., dams, embankments, roads, buildings and bridges) and the provision of services such as water supply and sewerage. Civil engineering projects involve the use of various materials for design and construction. It is commonly expected that civil engineers have an in-depth knowledge of these conventional and advanced materials to select the materials sensibly, determine the material properties, and effectively carry out the design and construction.

Civil Engineering Materials covers all major traditional civil engineering materials through separate chapters. Sustainability is an important consideration these days among civil and construction engineering professionals. It is and will be a critical element in the material selection process. Chapter 13 discusses the sustainability, life cycle analysis, and other important issues relevant to civil and construction materials.

The civil engineering materials course is almost always taught in a broad-brush approach rather than providing comprehensive coverage. Generally, students learn the subject in the early years of a civil engineering program, and with the content covering a wide range of rather independent topics, it is necessary to provide a broad-brush treatment without relying on too many prerequisites. The students subsequently take more detailed courses in soils, rocks, concrete, steel, etc. that offer higher-level coverage.

All four co-authors are passionate about what we do in our respective areas, with excellent track records in teaching and learning. We are also active researchers who are up-to-date with the recent developments. Being young or young at heart, we have a good blend of experience; the young ones are eager to provide good taste and layout that is appealing to the new generation, and the senior ones take charge and contribute through their experiences. Having four co-authors with different backgrounds is one of the strengths here. This has enabled us to develop the chapters with genuine expertise in areas that we have been teaching for years.

Organization

Each chapter covers a single topic thoroughly, so that instructors can change the sequence and select chapters for their teaching needs.

Chapter 1 Engineering Behavior of Materials—Some Fundamentals introduces the study of civil engineering materials and covers some basic concepts such as loads, stress, strain, constitutive models, and measurements.

Chapter 2 Chemistry of Materials provides an overview of the fundamental chemical aspects of materials at an atomic level. It also explains how materials are classified based off of their chemical structure.

Chapters 3 through 12 describe the properties and uses of various materials, including relevant information specific to each material. The following materials are presented:

Chapter 3 Soils
Chapter 4 Rocks
Chapter 5 Aggregates
Chapter 6 Geosynthetics
Chapter 7 Asphalt Cement and Hot Mix Asphalt Concrete
Chapter 8 Cement and Concrete
Chapter 9 Metals and Alloys
Chapter 10 Steel
Chapter 11 Polymers, Ceramics, and Composites
Chapter 12 Wood

Chapter 13 Sustainability of Construction Materials takes a look at issues of sustainability in the context of civil engineering, discussing the importance of material selection, conducting life cycle analyses, and other practices necessary to consider for present and future engineering challenges.

Key Features

Simple explanations and clarity of approach are main strengths of this book, making it very reader friendly. A wealth of worked examples and end-of-chapter exercises cover different difficulty levels. The content is covered in an applied manner, relating to real-world engineering issues, the book provides adequate theoretical background in every chapter. With the focus on materials and their applications, the visuals in the book are in full-color to help students see and understand how these materials work.

The organization of the book lends itself to easy course customization. Most of the chapters on individual materials are independent which gives the instructor freedom to change the order and even omit a chapter or two as appropriate to their subject/course outline. This also allows students using it as a reference to easily find the information they need on each topic.

Coverage of sustainability is one of the highlights of this book. There is an increased emphasis placed in civil engineering practice on sustainability in material selection in the designs and construction. Starting to think about sustainability early during the first years of their civil engineering training helps students lay the groundwork as their studies and work continues.

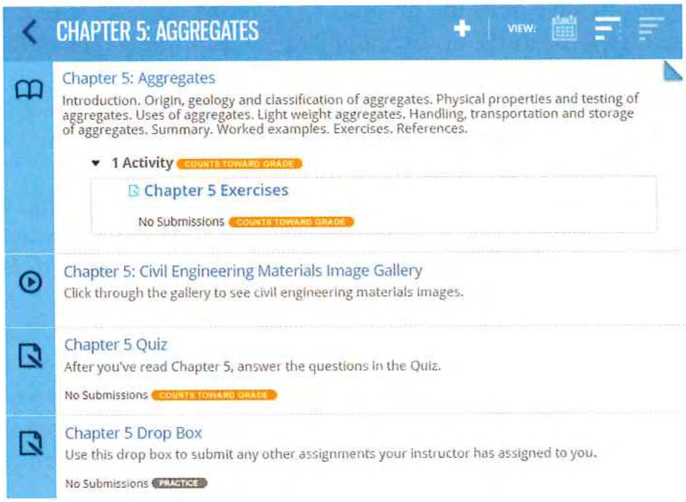
Supplements

A detailed Instructor's Solutions Manual, containing solutions to all problems from the text, and Lecture Note PowerPoint Slides are available via a secure, password-protected Instructor Resource Center at http://login.cengage.com.

MindTap Online Course

Civil Engineering Materials is also available through **MindTap**, Cengage Learning's digital course platform. The carefully crafted pedagogy and exercises in this textbook are made even more effective by an interactive, customizable eBook, automatically graded assessments, and a full suite of study tools.

As an instructor using MindTap, you have at your fingertips the full text and a unique set of tools, all in an interface designed to save you time. MindTap makes it easy for instructors to build and customize their course, so you can focus on the most relevant material while also lowering costs for your students. Stay connected and informed through real-time student tracking that provides the opportunity to adjust your course as needed based on analytics of interactivity and performance. **End-of-chapter assessments** test students' knowledge of topics in each chapter. **A curated collection of civil engineering images** helps students better understand key concepts as they progress through the course.



CHAPTER 5: AGGREGATES

Chapter 5: Aggregates

Introduction. Origin, geology and classification of aggregates. Physical properties and testing of aggregates. Uses of aggregates. Light weight aggregates. Handling, transportation and storage of aggregates. Summary. Worked examples. Exercises. References.

1 Activity COUNTS TOWARD GRADE

Chapter 5 Exercises

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Chapter 5: Civil Engineering Materials Image Gallery

Click through the gallery to see civil engineering materials images.

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After you've read Chapter 5, answer the questions in the Quiz.

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Nagaratnam Sivakugan
Carthigesu T. Gnanendran
Rabin Tuladhar
Bobby Kannan Mathan

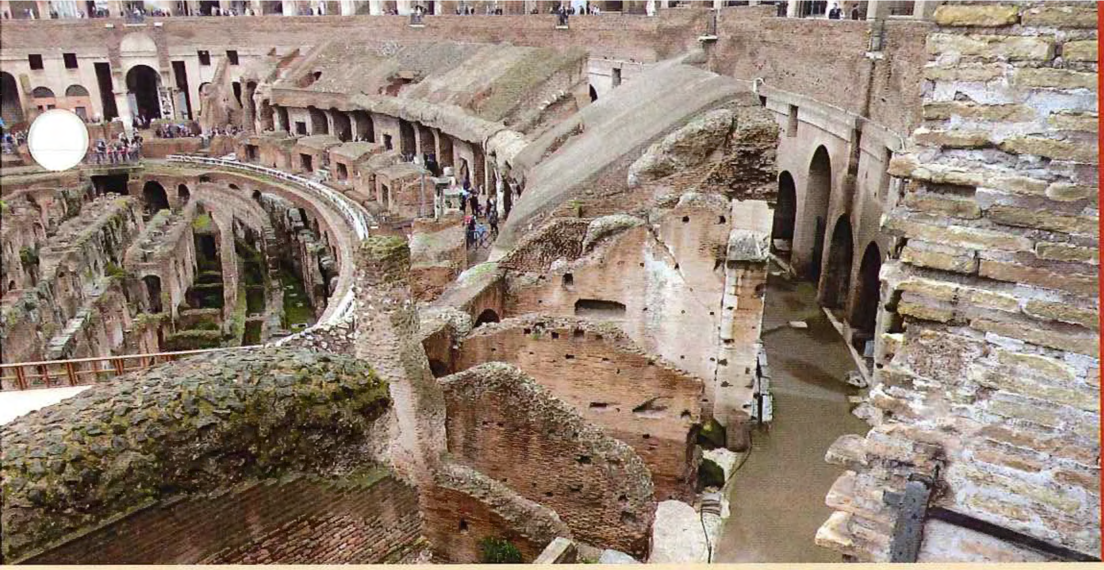
ABOUT THE AUTHORS

Dr. Sivakugan received his Bachelor's degree in Civil Engineering from University of Peradeniya, Sri Lanka, with First Class Honors. He earned his MSCE and PhD from Purdue University, West Lafayette, Indiana. Dr. Sivakugan's writings include six books, 130 international journal papers, 90 international conference papers, and more than 100 consulting reports. As a registered professional engineer of Queensland and a chartered professional engineer, Dr. Sivakugan does substantial consulting work for the geotechnical and mining industry in Australia and around the the world, including the World Bank. He is a Fellow of the American Society of Civil Engineers and Engineers Australia. He has supervised 13 PhD students to completion at James Cook University, Queensland, Australia, where he was the Head of Civil Engineering from 2003 to 2014. He is the Associate Editor of *International Journal of Geotechnical Engineering* and *International Journal of Geosynthetics and Ground Engineering*, and an editorial board member of *Canadian Geotechnical Journal* and *Indian Geotechnical Journal*.

Dr. Gnanendran received his Bachelor's degree in Civil Engineering from University of Peradeniya, Sri Lanka; an MEng from Carleton University, Canada; and a PhD from The University of Western Ontario, Canada. He has published over 100 scholarly articles in journals and international conferences and over 50 of them were in high-impact peer reviewed journals. He is an Associate Editor of the *Canadian Geotechnical Journal* and a member of the editorial board member of the *Geotextiles and Geomembranes Journal*. He has extensive industrial and high-level consulting experience for over 20 years and is a Fellow of Engineers Australia and a Chartered Professional Engineer in Australia. He has supervised 14 PhD students and 4 ME (research) students to successful completion.

Dr. Tuladhar received his Bachelor's degree in Civil Engineering from Regional Engineering College Silchar, India, with First Class Honors. He finished his Masters in Structural Engineering from Asian Institute of Technology, Thailand and PhD in Structural Engineering from Saitama University, Japan. He is currently a Senior Lecturer and Associate Dean Learning and Teaching at the College of Science and Engineering, James Cook University. He was awarded Australian Learning and Teaching Citation Award in 2011 for the outstanding contribution to student learning. He is also the winner of the Australian Innovation Challenge 2015 under Manufacturing, Construction and Infrastructure category for the development of recycled plastic fiber to reinforce concrete.

Dr. Mathan received his PhD in Materials Engineering from the Indian Institute of Technology Bombay (IITB) in India. He is currently an Associate Professor at James Cook University (JCU). He was the Head of the Chemical Engineering Department at JCU from 2010 to 2014. He has published over 90 scientific articles, including seven book chapters in the field of materials engineering. He is a Key Reader for *Metallurgical and Materials Transactions* (Springer), an editorial board member for two international journals, and an ad hoc reviewer for over 30 international journals.



CHAPTER 1

Civil engineering materials have been used since the ancient times. Several materials were used to build the Colosseum including travertine (a type of limestone), tuff (a porous rock made of volcanic ash), and concrete. Courtesy of N. Sivakugan, James Cook University, Australia

Engineering Behavior of Materials—Some Fundamentals

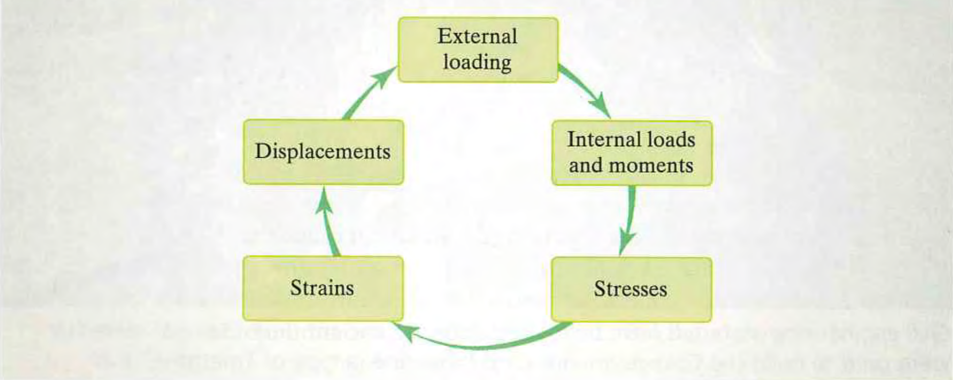
1.1 Introduction

Civil engineers design and construct buildings, bridges, dams, tunnels, highways, airports, canals, and play key roles in all infrastructure developments. Their work is not just limited to land; there is substantial offshore work going on worldwide. They also contribute in the mining industry, where the ore is removed from underground and minerals are extracted. Yes, they make their mark above the ground, below the ground, and out into the sea!

Civil engineers deal with a wide range of materials including, concrete, steel, soils, rocks, aggregates, timber, metals, and plastics. All structures have to be founded on the ground, and this requires a good understanding of soils, rocks, and aggregates. Concrete and steel are the two of the most used materials in civil engineering for construction of buildings and bridges. In spite of its lower load-carrying capacity compared to steel or concrete and shrinking supplies worldwide, timber is still being used as a building material, especially in housing where the loadings are relatively small in magnitude. Aggregates, asphalt and bitumen are used widely in roadwork. Geosynthetics are becoming increasingly popular in roadwork and slopes, and new products are coming into the market every year. Metals, polymers, and composites also have their place in civil engineering construction.

Engineering materials, or specifically civil engineering materials, is presented early in most curriculums at the university level. This is followed by subjects that cover certain materials (e.g., soils, concrete) to a much greater depth. Some materials, such as geosynthetics or rocks, may be covered more under electives, and some (e.g., metals, alloys, polymers) are covered only in civil engineering materials courses. Mechanical engineers may focus on metals and alloys more than concrete, soil, or timber. A good understanding of material behavior enables us to make the best use of them in designs and construction.

Figure 1.1 Loads—stresses—strains—displacements sequence



Very often, the site conditions, the loadings, and the materials are so complex that it is necessary to simplify the problem into something that is solvable. This requires certain assumptions and simplifications. Determining the design parameters, including the loadings and the material characteristics, is not always straightforward. Some simple *material models*, also known as *constitutive models*, commonly used to describe civil engineering materials, are described in the next section. They are often approximations of the actual material behavior but greatly simplify the problem without compromising too much on the results.

In general, the external forces (e.g., wind loads, live loads) produce internal forces and moments within the material, which in turn induce internal stresses. These internal stresses produce strains within the material, which result in displacements. This sequence is illustrated in Figure 1.1.

All civil engineering projects have a client, consultant, and a contractor. These three can be government organizations, private enterprises, or individuals. The client is the one who requires the development of a facility such as a building, bridge, or dam, has the funds to pay all associated costs, and owns the finished product. Consultants are professional engineers who the client hires to provide advisory service throughout the project, including analyses, designs, and specifications that constitute the basis of the construction contract. Contractors are the ones who undertake the real construction, based on the given designs and specifications. They need to commit on time frames, and there can be hefty penalties for delays.

Of late, this traditional tripartite relationship has been undergoing considerable change as many new forms of relationships are being considered, including alliance projects, design-and-build projects, turnkey projects, and so forth, where the consultant and contractor form an alliance with close collaboration in design and construction stages. Terms of engagement are also significantly different in such contracts.

The contract documents clearly specify the roles and obligations of all parties concerned, the method of assuring the quality of the work, the time frames and milestones, and points at which partial payments are made. In ensuring that the structure or facility meets its intended purpose, every component has to meet certain performance criteria. To this end, one of the key requirements is to ensure that the materials used in the project meet the specifications.

1.2 Stress-Strain Relations and Constitutive Models

A successful civil engineer must have a good understanding of statics, which enables one to isolate a certain section of the problem, examine the free-body diagram, apply equilibrium equations, and determine the unknown forces. In addition, it is necessary to have a good understanding of how the different materials behave under a wide range of loading conditions (e.g., tension, compression, and shear). These are discussed more in the engineering strength of materials course. A *constitutive model* is simply a *material model* that describes mathematically how the stresses and strains are related. Under the same loadings, the strains and displacements can be quite different, depending on the material (i.e., constitutive model). Before attempting to carry out any computations of strains or deformations, it is necessary to define the constitutive model for the material.

Figure 1.2 shows a simply supported beam subjected to a point load P. The reactions at the support R_1 and R_2 can be determined from equilibrium considerations as

$$R_1 = \frac{P l_2}{l_1 + l_2} \tag{1.1}$$

$$R_2 = \frac{P l_1}{l_1 + l_2} \tag{1.2}$$

No consideration is given to the material properties in computing the reactions R_1 and R_2. They would have been the same irrespective of whether the beam was made of steel, concrete, or timber. This is true for bending moments and shear forces, too. However, if the deflections are required, the material properties will come into play.

Figure 1.3 shows the stress-strain diagram of a cylindrical specimen of length l and cross-sectional area A, subjected to a tensile axial load P. Here, the axial stress σ and strain ε are given by

$$\sigma = \frac{P}{A} \tag{1.3}$$

and

$$\varepsilon = \frac{\Delta l}{l} \tag{1.4}$$

A simply supported beam subjected to a point load P

Figure 1.2

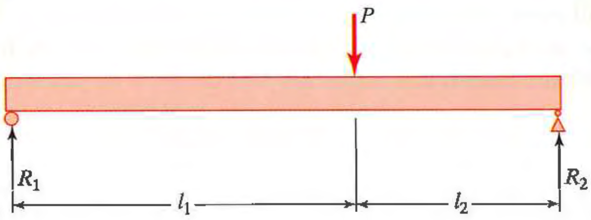
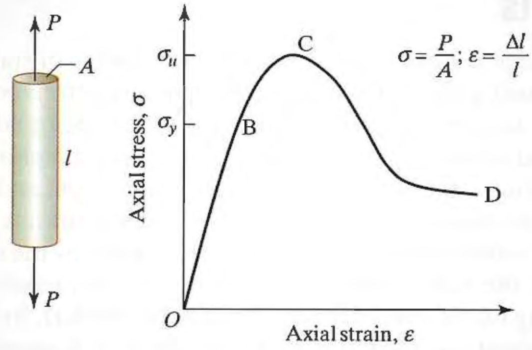


Figure 1.3 Stress-strain behavior of a material



where Δl is the elongation under the load P. In the segment OB of the stress-strain (σ-ε) diagram, the axial stress is proportional to the axial strain, where the material is behaving as a *linear elastic material*, following Hooke's law. At B, when the normal stress exceeds σ_y, the material loses the linearity where the yielding takes place, and the material undergoes some irrecoverable *plastic strains*. The value σ_y is known as the *yield stress* of the material, which is a unique property of the material that does not depend on the specimen dimensions. The *ultimate stress* σ_u is the maximum axial stress the material can withstand, which is reached at C. Beyond C, lesser load is required to deform the material, which exhibits *strain-softening behavior* and fails at D.

1.2.1 Some Simple Material Models

The simplest of all material models is the linear elastic model, proposed by the English scientist Robert Hooke (1635–1703), and hence is known as *Hooke's law*. Here, the stress-strain diagram, as shown in Figure 1.4a is a straight line. At any stage of loading, if the load is removed, the object returns to its original state, following the same loading path in the σ-ε diagram without any sign of permanent deformation. This is known as elastic behavior. The slope of the σ-ε plot (see Figure 1.4a) is known as the *modulus of elasticity* or *Young's modulus*, named after the English scientist Thomas Young (1773–1829). The Young's modulus can be in a very wide range, depending on the material. The softer the material, the lower the modulus is. It is a measure of the material's resistance to deformation. Table 1.1 summarizes the Young's modulus of some typical civil engineering materials.

At any stage of loading, the stress and strain are related by the following expression in a linear elastic material.

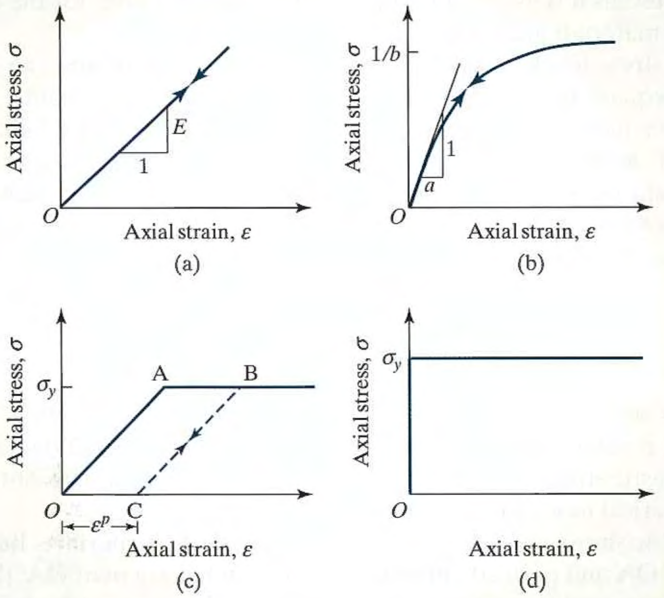
$$\sigma = E\,\varepsilon \tag{1.5}$$

Any changes in the longitudinal stress and strain, denoted here as $\Delta\sigma$ and $\Delta\varepsilon$, respectively, are related by

$$\Delta\sigma = E\,\Delta\varepsilon \tag{1.6}$$

Some simple constitutive models: (a) linear elastic, (b) nonlinear elastic, (c) elastic-plastic, and (d) rigid plastic

Figure 1.4



The lateral normal strain and the longitudinal (i.e., axial) normal strain are related by

$$\nu = -\frac{\text{lateral normal strain}}{\text{axial normal strain}} \tag{1.7}$$

Young's modulus of typical civil engineering materials

Table 1.1

Material	Young's modulus, E (GPa and kip/ft^2)	Poisson's ratio, ν
Soils	0.01–0.1 (210–2100)	0.1–0.5
Plastics	1–3 (21–63 × 10^3)	0.4
Wood	10–15 (210–313 × 10^3)	NA*
Concrete	20–30 (420–630 × 10^3)	0.1–0.2
Rocks	10–100 (210–2100 × 10^3)	0.2–0.3
Aluminum alloys	73 (1525 × 10^3)	0.33
Brass	100 (2100 × 10^3)	0.34
Copper and copper alloys	110–120 (2300–2500 × 10^3)	0.33–0.36
Cast iron	80–170 (1670–3550 × 10^3)	0.2–0.3
Steel	200 (4180 × 10^3)	0.27–0.30

* *Variable depending on the direction of loading and other parameters.*

Note: kip/ft^2 in parentheses.

where ν is known as the *Poisson's ratio*, named after Siméon Denis Poisson (1781–1840), which is constant to a specific material. It is denoted by the Greek symbol ν (nu) or sometimes μ (mu). Poisson's ratio lies in the range of 0–0.5, and for most materials it is in the range of 0.25–0.35. Typical values for the common civil engineering materials are given in Table 1.1.

At low stress levels, many materials behave elastically, and the linear elastic model is adequate to describe their stress-strain behavior. A slightly superior but more complex model is a *nonlinear elastic model* shown in Figure 1.4b. At any stage of loading, if the load is removed, the material returns to its original shape without undergoing any permanent or plastic deformation, following the same nonlinear path in the σ-ε diagram. One of the simplest nonlinear elastic models can be described by the following expression.

$$\sigma = \frac{\varepsilon}{a + b\varepsilon} \tag{1.8}$$

where $1/a$ is the slope of the σ-ε diagram at zero strain, and $1/b$ is the ultimate stress, which is reached at a very large strain (see Figure 1.4b). Known as hyperbolic nonlinear elastic stress-strain model, this is one of the popular constitutive models used in numerical modeling work.

Figure 1.4c shows an *elastic-plastic model*, where the material is linear elastic in the segment OA and perfectly plastic beyond A. In the segment OA, the strains are elastic and recoverable. Any unloading will bring the material back to its original state. The material yields at A and behaves as a *perfectly plastic material* from there. Any further deformation or strain beyond A is entirely plastic, which is irrecoverable. When the material is unloaded from B, it follows a path BC parallel to the elastic segment OA. When fully unloaded to C, the material has undergone a permanent, irrecoverable plastic strain, denoted by ε^p. When reloaded, the material behaves linear elastically, following the same straight-line path until it reaches the new yield point B, at the same yield stress σ_y.

Figure 1.4d shows a rigid plastic model, where the material remains rigid without any strains or deformations until the yield stress is reached. On reaching the yield stress, the material behaves plastically and undergoes very large deformation.

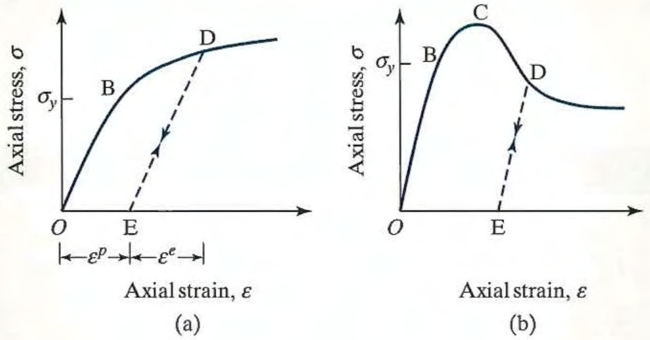
1.2.2 Other Material Models

In reality, no materials would strictly follow any of the models discussed in Section 1.2.1. Nevertheless, these models are relatively easier to implement in computing the strains and deformations. Figure 1.5 shows *strain-hardening* and *strain-softening* models where the material behaves elastically until it reaches the yield stress σ_y at B. When the material is unloaded at D, it follows the straight-line path DE. When reloaded from E, it follows the same path ED and then continues along the original σ-ε curve. In the case of a strain-hardening material (Figure 1.5a), the yield stress has increased by loading beyond the original yield stress and unloading. In strain-softening model shown in Figure 1.5b, the yield stress is reduced by straining further.

Generally, elastic and plastic strains occur simultaneously in most materials, and these models are known as *elasto-plastic models*, which are different from the simpler elastic-plastic model described in Figure 1.4c. In the strain-hardening model shown in Figure 1.5a, the axial strain ε at D consists of elastic and plastic components ε^e and ε^p, respectively such that $\varepsilon = \varepsilon^e + \varepsilon^p$.

Other stress-strain diagrams: (a) strain hardening, and (b) strain softening

Figure 1.5



Mohr-Coulomb and Drucker-Prager are two of the commonly used elasto-plastic models. There are other material models that suit specific material under specific loading conditions.

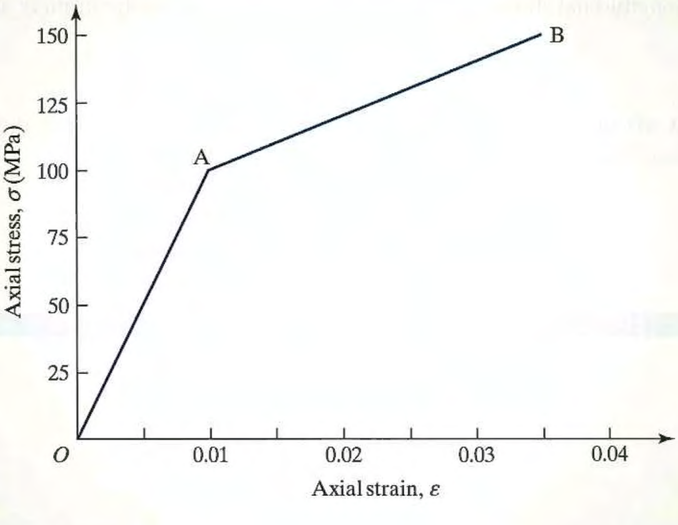
EXAMPLE 1.1

The stress-strain plot of a material is shown in Figure 1.6, where segment OA is linear elastic. It can be assumed that unloading (and reloading) anywhere in segment AB will take place along a path parallel to OA.

a. Determine the Young's modulus of the material.
b. Find the elastic and plastic strains at B where the total axial strain is 0.035 (or 3.5%) and the axial stress is 150 MPa.
c. What is the elastic strain during deformation from A to B?

Stress-strain diagram for Example 1.1

Figure 1.6



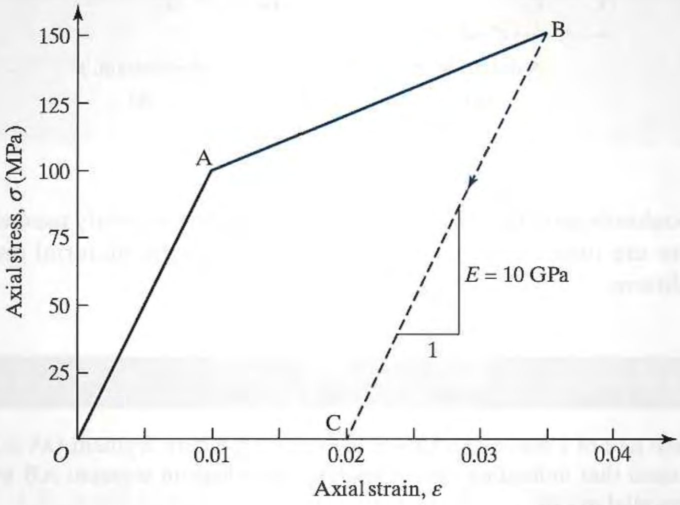
Solution:

a. In segment OA, the Young's modulus is given by the slope of the line. Hence,

$$E = \frac{100}{0.01}\,\text{MPa} = 10\,\text{GPa}$$

b.

Figure 1.7 Calculation of elastic and plastic strains in Example 1.1



When the material is unloaded from B to C, the elastic strain is fully recovered. This elastic strain is given by

$$\varepsilon^e = \frac{150}{10{,}000} = 0.015$$

The total longitudinal strain ε at B $= 0.035$. Therefore the plastic strain ε^p is given by

$$\varepsilon^p = \varepsilon - \varepsilon^e = 0.035 - 0.015 = 0.020$$

c. In segment AB, the longitudinal stress is increased by 50 MPa. Therefore, the elastic strain during this segment is given by

$$\varepsilon^e = \frac{50}{10{,}000} = 0.005$$

1.3 Types of Loadings

The loadings on a system (e.g., a building) can include forces, pressures, stresses, moments, and torques. The forces can be compressive, tensile, or shear and are usually denoted by uppercase letters such as *F, P, Q, R, S,* and *T,* with units of N, kN or MN (lb, kip or ton). The stresses can be compressive, tensile, or shear. Tensile and compressive stresses act normal to the surface and are often denoted by the Greek symbol σ (sigma), and the shear stresses act along the surface and are denoted by the Greek symbol τ (tau). Common units for stresses are N/m^2 (or pascal, written as Pa), kN/m^2, or MN/m^2 (or lb/ft^2, lb/in^2, kip/ft^2, ton/ft^2). Very often the forces would be acting at an angle to the surface that can be resolved into a normal and shear loads, producing a normal stresses and shear stresses. *Pressure* is a normal stress, and examples include atmospheric pressure, air pressure, hydrostatic pressure in water, and so forth.

When a beam is subjected to loads in the transverse direction (i.e., normal to the longitudinal axis) as shown in Figure 1.2, it *bends* and produces *bending moments* which induce *bending stresses* throughout the beam. When a tall building or a transmission tower is subjected to lateral wind loads, large moments are induced. The axis of the moment is perpendicular to the plane of bending. The bending moment is the product of force and distance, and hence has the unit of N·m, kN·m or MN·m (lb·ft, kip·ft or ton·ft). The bending stress σ at distance y from the neutral axis can be determined by

$$\sigma = \frac{M}{I}y \tag{1.9}$$

where M = moment at the section, and I = moment of inertia about the neutral axis. The shear stress τ at any location on the cross section is given by

$$\tau = \frac{VQ}{It} \tag{1.10}$$

where V = shear force at the section, Q = first moment of the cross sectional area above the level at which shear stress is being determined, and t = thickness of the section at the level where the shear stress is being determined.

Torque is a *twisting moment* applied to a longitudinal bar in an attempt to rotate the bar about its longitudinal axis. It is similar to moment, but the axis of the torque is longitudinal. The units are the same as for moments. A torque T applied to a shaft produces shear stresses which can be determined by

$$\tau = \frac{T\rho}{I_P} \tag{1.11}$$

where ρ = distance from the center where the shear stress is being determined, and I_P = polar moment of inertia of the cross section. Eqs. 1.9 to 1.11 are generally discussed in detail in strength of materials subject.

1.4 Special Loading Situations

All design problems, including buildings, bridges, dams, and foundations, are three-dimensional in nature. The loads and deformations, and the resulting stresses and strains, can be in any direction in a three-dimensional space. Sometimes, for the sake of simplicity, the stresses and/or strains in the third dimension are ignored in the analysis, and the problems are assumed to be two-dimensional. Some of these special cases include axisymmetric, plane strain, and plane stress loadings. They are discussed in this section.

Sign Convention

The *right hand Cartesian coordinate system* is the most commonly used mnemonic for defining the directions of x, y, and z axes in three dimensions. When the index finger, thumb, and the middle finger of the right hand are held perpendicular to each other, they can define the relative directions of the three mutually perpendicular axes of a coordinate system. If the thumb points along the x-axis and the index finger points along the y-axis, the middle finger points along the z-axis.

As shown in Figure 1.8a, the positive x-plane is the plane where the normal to the plane points along the positive x-axis. The negative x-plane is the one where the normal to the plane points in the opposite direction of the x-axis. This applies to all three directions. The sign convention commonly used in applying the equilibrium equations relating to stresses at a point is as follows:

- The shear or normal stress is *positive* when it acts on a positive plane and points in the positive direction, or when it acts on a negative plane and points in the negative direction.
- The shear or normal stress is *negative* when it acts on a positive plane and points in the negative direction, or when it acts on a negative plane and points in the positive direction.

Figure 1.8 Sign convention in the three-dimensional Cartesian coordinate system: (a) positive directions and planes, and (b) shear and normal stresses at a point

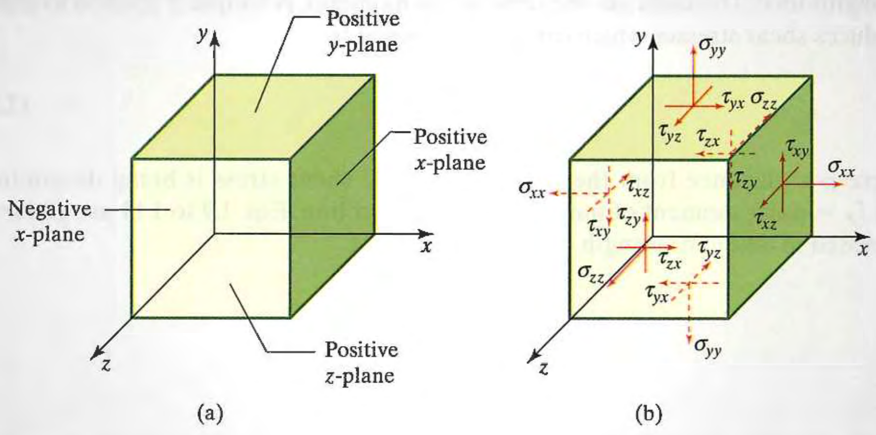


Figure 1.8b shows the shear and normal stresses acting on the six sides of an infinitesimally small element, representing a point. The body forces (e.g., self-weight) are neglected here. It is worth noting that all the normal and shear forces marked here are *positive* (based on the sign convention). The first subscript after the symbols σ and τ defines the plane on which the stress is acting, and the second subscript indicates the direction of the stress. For example, τ_{xy} is the shear stress acting on x-plane in y-direction. From equilibrium considerations, $\tau_{xy} = \tau_{yx}$, $\tau_{yz} = \tau_{zy}$, and $\tau_{zx} = \tau_{xz}$.

It is important to have a good grasp of the sign convention when interpreting the directions of stresses determined from equations that were developed from the above and computer program output files.

When the problem is approximated as a two-dimensional problem, it can be represented in Cartesian or *cylindrical polar coordinate system.* Figure 1.9 shows the stresses on an infinitesimal element, in a two-dimensional Cartesian coordinate system. As before, $\tau_{xy} = \tau_{yx}$. The second subscript is often omitted for the normal stresses, which are simply denoted by σ_x, σ_y, and σ_z.

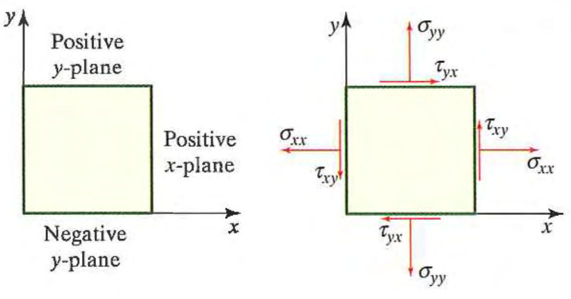
Stress Transformation and Principal Stresses

It is worth emphasizing that the state of stress represented by the element in Figure 1.9 (also in Figure 1.8) refers to a point. Using two-dimensional stress-transformation equations or Mohr circles introduced in a strength of materials course, the stresses in any other direction can be determined by rotating the element appropriately.

Principal stresses are the values of normal stresses when the shear stresses acting on the planes are zero. The corresponding planes are known as *principal planes*. The largest and the smallest values of the principal stresses are known as *major principal stress* σ_1 and *minor principal stress* σ_3, respectively. The notation σ_2 is reserved for the *intermediate principal stress* that acts in the direction perpendicular to the plane containing σ_1 and σ_3. Similarly, there are also *principal strains* $\varepsilon_1, \varepsilon_2,$ *and* ε_3.

Stresses on a two-dimensional element

Figure 1.9



1.4.1 Generalized Stress-Strain Relationships

The generalized stress-strain relationship for a material can be expressed in a matrix form as

$$\begin{pmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{pmatrix} = \begin{pmatrix} C_{11} & C_{12} & C_{13} & C_{14} & C_{15} & C_{16} \\ C_{21} & C_{22} & C_{23} & C_{24} & C_{25} & C_{26} \\ C_{31} & C_{32} & C_{33} & C_{34} & C_{35} & C_{36} \\ C_{41} & C_{42} & C_{43} & C_{44} & C_{45} & C_{46} \\ C_{51} & C_{52} & C_{53} & C_{54} & C_{55} & C_{56} \\ C_{61} & C_{62} & C_{63} & C_{64} & C_{65} & C_{66} \end{pmatrix} \begin{pmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{pmatrix} \tag{1.12}$$

where γ_{xy}, γ_{yz}, and γ_{zx} are the shear strains in the xy, yz and zx planes. The 6×6 matrix in Eq. 1.12 is known as the stiffness matrix, which has 36 *constants of proportionality*, known as *elastic constants*. Generally, due to symmetry (i.e., $C_{12} = C_{21}$), there are only 21 independent constants. When the material is assumed to be homogeneous and *isotropic*, the number of independent elastic constants reduces to just two. Five of the most common material constants used to describe a homogeneous isotropic elastic material, where only two are independent, are Lame's constant (λ), Young's modulus (E), Poisson's ration (ν), shear modulus (G), and bulk modulus (K).

Anisotropic materials are ones where the properties vary with the direction. *Orthotropic* materials (e.g., wood) are those with two or three orthogonal planes of symmetry, showing different mechanical properties along each of the three axes. They have nine independent elastic constants. *Transversely isotropic materials* (sometimes known as *cross anisotropic* materials) are the ones that demonstrate symmetry about an axis, which is normal to the planes of isotropy. They have five independent elastic constants. Soils are commonly assumed to be cross anisotropic, where the plane of isotropy is horizontal. The properties in the isotropic plane will be different to those in the direction perpendicular to the plane. Isotropic materials are those with the same properties in all directions.

Isotropic Linear Elastic Material

The interrelationships between the six stress components and the six strain components of an isotropic linear elastic material can be written as:

$$\varepsilon_x = \frac{1}{E}[\sigma_x - \nu(\sigma_y + \sigma_z)] \tag{1.13a}$$

$$\varepsilon_y = \frac{1}{E}[\sigma_y - \nu(\sigma_x + \sigma_z)] \tag{1.13b}$$

$$\varepsilon_z = \frac{1}{E}[\sigma_z - \nu(\sigma_x + \sigma_y)] \tag{1.13c}$$

$$\gamma_{xy} = \frac{1}{G}\tau_{xy} \tag{1.13d}$$

$$\gamma_{yz} = \frac{1}{G}\tau_{yz} \tag{1.13e}$$

$$\gamma_{xz} = \frac{1}{G}\tau_{xz} \tag{1.13f}$$

where E and G are the Young's modulus and shear modulus (or modulus of rigidity) respectively. E and G are related by:

$$G = \frac{E}{2(1+\nu)} \tag{1.14}$$

In matrix form, Eq. 1.13 can be represented as:

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{Bmatrix} = \frac{1}{E}\begin{bmatrix} 1 & -\nu & -\nu & 0 & 0 & 0 \\ -\nu & 1 & -\nu & 0 & 0 & 0 \\ -\nu & -\nu & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 2(1+\nu) & 0 & 0 \\ 0 & 0 & 0 & 0 & 2(1+\nu) & 0 \\ 0 & 0 & 0 & 0 & 0 & 2(1+\nu) \end{bmatrix}\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{Bmatrix} \tag{1.15}$$

or

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)}\begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ \nu & 1-\nu & \nu & 0 & 0 & 0 \\ \nu & \nu & 1-\nu & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{(1-2\nu)}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{(1-2\nu)}{2} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{(1-2\nu)}{2} \end{bmatrix}\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{Bmatrix} \tag{1.16}$$

The volumetric strain ε_{vol} is the ratio of volume change to the initial volume and is given by:

$$\varepsilon_{\text{vol}} = \varepsilon_x + \varepsilon_y + \varepsilon_z \tag{1.17}$$

Substituting the expressions for strains from Eq. 1.13,

$$\varepsilon_{\text{vol}} = \frac{1-2\nu}{E}[\sigma_x + \sigma_y + \sigma_z]$$

$$\varepsilon_{\text{vol}} = \frac{3(1-2\nu)}{E}\frac{[\sigma_x + \sigma_y + \sigma_z]}{3} = \frac{1}{K}\frac{[\sigma_x + \sigma_y + \sigma_z]}{3} \tag{1.18}$$

where K is the *bulk modulus*, given by

$$K = \frac{E}{3(1 - 2\nu)} \tag{1.19}$$

In some numerical modeling applications, G and K are used as input parameters, rather than E and ν. They are related by:

$$E = \frac{9KG}{3K + G} \tag{1.20}$$

$$\nu = \frac{3K - 2G}{2(3K + G)} \tag{1.21}$$

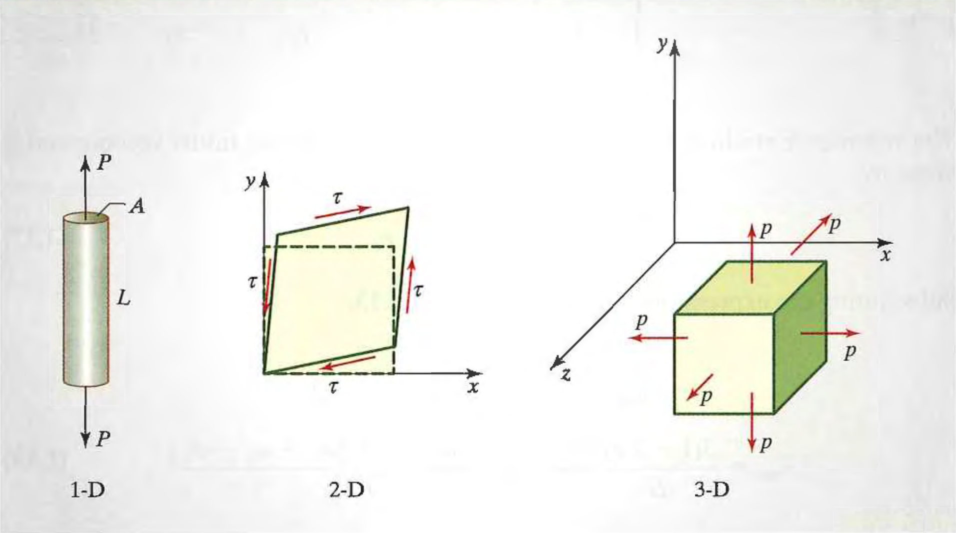
E, G, and K are measures of material stiffness. The definitions of E, G, and K are reinforced through Figure 1.10.

In a bar subjected to axial load P, the Young's modulus of a linear elastic isotropic material is given by $E = \sigma/\varepsilon$, where σ and ε are the axial stress and strain, respectively. In the two-dimensional plane element subjected to pure shear in Figure 1.10, the shear modulus G is defined as $G = \tau/\gamma$, where τ and γ are the shear stress and shear strain, respectively. In three dimensions, when an element of volume V is subjected to equal tensile stresses in all directions, the increase in volume is ΔV. Here, the *volumetric strain* is given by

$$\varepsilon_{\text{vol}} = \frac{\Delta V}{V} \tag{1.22}$$

and the bulk modulus K is defined as $K = p/\varepsilon_{\text{vol}}$.

Figure 1.10 Definitions of E, G, and K



EXAMPLE 1.2

Deduce that the Poisson's ratio is 0.5 for an elastic incompressible material.

Solution:

Incompressible material implies that the volumetric strain is zero when subjected to any normal stress (It can undergo distortion in the shape).

That means the bulk modulus K must be infinity (i.e., very large).

$$K = \frac{E}{3(1 - 2\nu)} \tag{1.19}$$

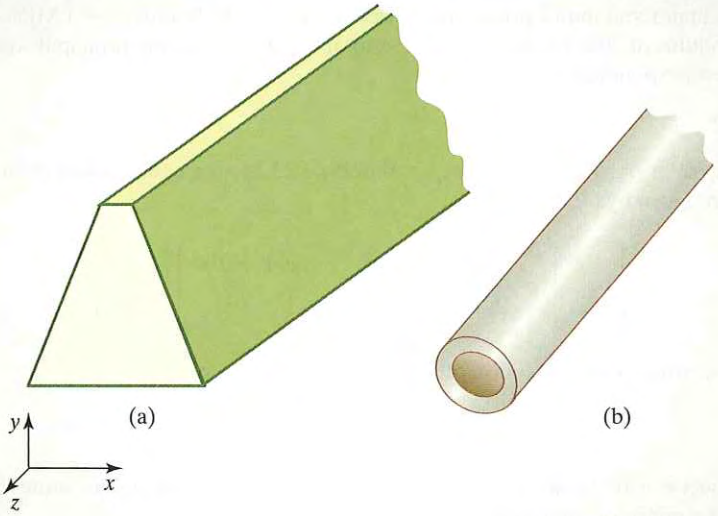
From Eq. 1.19, for K to be infinity, $\nu = 0.5$.

1.4.2 Plane Strain Loading

In civil engineering, when the structure (e.g., wall, embankment, strip footing) is long in one direction, the deformation or strain in this longitudinal direction can be neglected, and the situation can be assumed as a *plane strain* problem. All deformations and strains are limited to the transverse plane that is normal to the longitudinal direction. Figure 1.11a shows a very long gravity retaining wall where the cross section is in the xy plane. Figure 1.11b shows a very long cylindrical pressure vessel with both ends closed and subjected to an internal pressure. In both these situations, at any transverse section perpendicular to the z-axis, the strains are only along the xy plane.

Figure 1.11

Examples of plane strain loading: (a) long retaining wall and (b) a pressure vessel with closed ends



(a) (b)

In plane strain loading situations, where the strains are limited to the xy plane, Eqs. 1.15 and 1.16 become:

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \frac{1}{E} \begin{bmatrix} 1-\nu^2 & -\nu(1+\nu) & 0 \\ -\nu(1+\nu) & 1-\nu^2 & 0 \\ 0 & 0 & 2(1+\nu) \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} \tag{1.23}$$

and

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & 0 \\ \nu & 1-\nu & 0 \\ 0 & 0 & \dfrac{(1-2\nu)}{2} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} \tag{1.24}$$

Plane strain loading does not mean that there are no normal stresses in the direction perpendicular to the plane. Only the strains are zero in that direction. The normal stress in the direction perpendicular to the plane (in the direction of zero normal strain) is given by:

$$\sigma_z = \nu(\sigma_x + \sigma_y) \tag{1.25}$$

In plane strain loading, the nonzero stresses are $\sigma_x, \sigma_y, \sigma_z$, and τ_{xy}. The nonzero strains are ε_x, ε_y, and γ_{xy}.

EXAMPLE 1.3

In plane strain loading, when the applied normal stresses are principal stresses, derive the expressions for the major and minor principal strains. In a rock mass subjected to plane strain loading, the major and minor principal stresses are $\sigma_1 = 2$ MPa and $\sigma_3 = 1$ MPa. Assuming a Young's modulus of 20 GPa and Poisson's ratio of 0.2, determine the principal strains and the normal stress perpendicular to the plane.

Solution:

Substituting $\sigma_x = \sigma_1, \sigma_y = \sigma_3$, and $\tau_{xy} = 0$ in Eq. 1.23, the major and minor principal strains, ε_1 and ε_3, are given by:

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_3 \end{Bmatrix} = \frac{1}{E} \begin{bmatrix} 1-\nu^2 & -\nu(1+\nu) \\ -\nu(1+\nu) & 1-\nu^2 \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_3 \end{Bmatrix}$$

The principal stress in the direction of zero normal strain is given by:

$$\sigma_2 = \nu(\sigma_1 + \sigma_3)$$

This is not necessarily the intermediate principal stress. Depending on the values of ν, σ_1, and σ_3, this can be minor or intermediate principal stress.

Substituting the values,

$$\varepsilon_1 = \frac{1}{20}\{(1 - 0.2^2) \times 2 - 0.2 \times (1 + 0.2) \times 1\} = 0.084$$

$$\varepsilon_3 = \frac{1}{20}\{-0.2 \times (1 + 0.2) \times 2 + (1 - 0.2^2) \times 1\} = 0.024$$

For plane strain loading, $\varepsilon_2 = 0$. For principal planes, the shear strains are zero as well. The normal stress in the direction of zero normal strain is given by:

$$\sigma_2 = 0.2 \times (2 + 1) = 0.6\,\text{MPa}$$

1.4.3 Plane Stress Loading

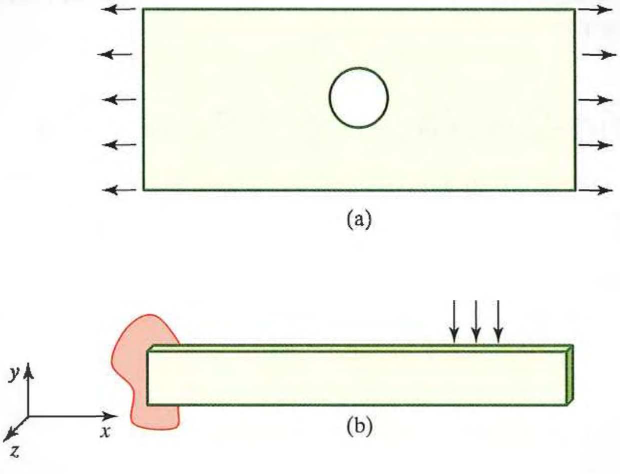
Figure 1.12a shows a thin plate with a hole subjected to external loads along its plane. Figure 1.12b shows a thin cantilever plate subjected to external loads. In both these situations, at any point on the plate, the stresses are acting only along the xy plane.

Let's consider a thin flat plate that is loaded along its plane. The stresses and the strains along the plane are related by:

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \frac{1}{E}\begin{bmatrix} 1 & -\nu & 0 \\ -\nu & 1 & 0 \\ 0 & 0 & 2(1+\nu) \end{bmatrix}\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} \tag{1.26}$$

Figure 1.12

Examples of plane stress loading: (a) a thin plate with a hole, loaded along its plane, and (b) a thin cantilever plate



or

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \frac{E}{(1-\nu)^2}\begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \dfrac{(1-\nu)}{2} \end{bmatrix}\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} \tag{1.27}$$

Here, the nonzero stresses in the three dimensional space are σ_x, σ_y, and τ_{xy}. There can be strains perpendicular to the xy plane, but not stresses. The nonzero strains are ε_x, ε_y, ε_z, and γ_{xy}. The normal strain in the direction of zero normal stress is given by:

$$\varepsilon_z = \frac{\nu}{1-\nu}(\varepsilon_x + \varepsilon_y) = -\frac{\nu}{E}(\sigma_x + \sigma_y) \tag{1.28}$$

1.4.4 Axisymmetric Loading

Axisymmetric loading is quite common in geotechnical engineering. For example, along the vertical center line of a uniformly loaded circular footing, the lateral stresses are the same in all directions. Generally, axisymmetric problems are studied using a polar-cylindrical coordinate system. If σ_1 and σ_3 are the axial and radial normal stresses respectively, they are related to the normal strains in the same directions ε_1 and ε_3 by:

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_3 \end{Bmatrix} = \frac{1}{E}\begin{bmatrix} 1 & -2\nu \\ -\nu & 1-\nu \end{bmatrix}\begin{Bmatrix} \sigma_1 \\ \sigma_3 \end{Bmatrix} \tag{1.29}$$

$$\begin{Bmatrix} \sigma_1 \\ \sigma_3 \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)}\begin{bmatrix} 1-\nu & 2\nu \\ \nu & 1 \end{bmatrix}\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_3 \end{Bmatrix} \tag{1.30}$$

The radial stress and the tangential (or circumferential) stress on a cylindrical element are equal along the axis of symmetry. It must be noted that Eqs. 1.29 and 1.30 are valid only along the center line, and the axisymmetric conditions do not apply outside the center line.

1.5 Strain-Displacement Relations

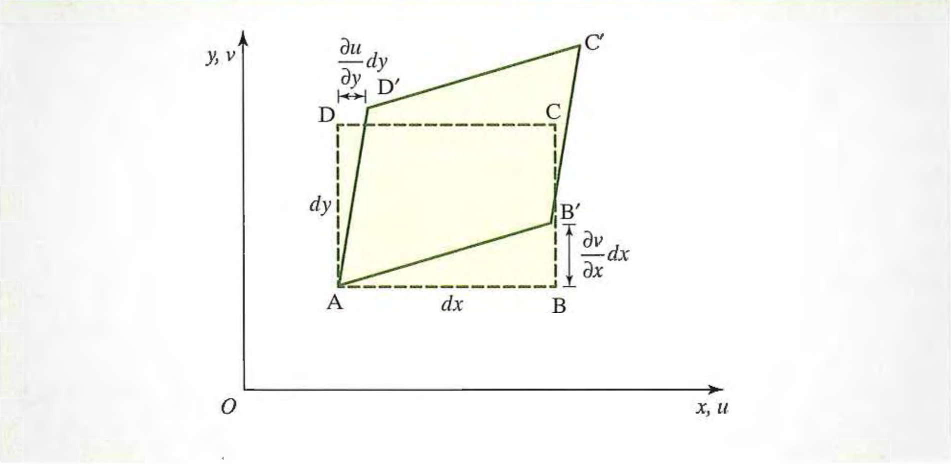
Figure 1.13 shows a differential element ABCD of dimensions dx and dy along the x and y directions, respectively, in a two-dimensional space. The displacements in x and y directions are denoted by u and v, respectively. The deformed element is shown as AB'C'D'. It can be shown that the normal strains in the x and y directions are given by

$$\varepsilon_x = \frac{\partial u}{\partial x} \tag{1.31}$$

$$\varepsilon_y = \frac{\partial v}{\partial y} \tag{1.32}$$

Differential element before and after deformation

Figure 1.13



The shear strain ε_{xy}, as defined in mechanics, is given by

$$\varepsilon_{xy} = \varepsilon_{yx} = \frac{1}{2}\left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}\right) \tag{1.33}$$

In engineering, the shear strain is defined a little differently. The *engineering shear strain* γ_{xy} is the change in angle DAB, which is given by

$$\gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} = 2\varepsilon_{xy} \tag{1.34}$$

For a three-dimensional loading, with an additional component of displacement w in z-direction, the additional strain components can be expressed as

$$\varepsilon_z = \frac{\partial w}{\partial z} \tag{1.35}$$

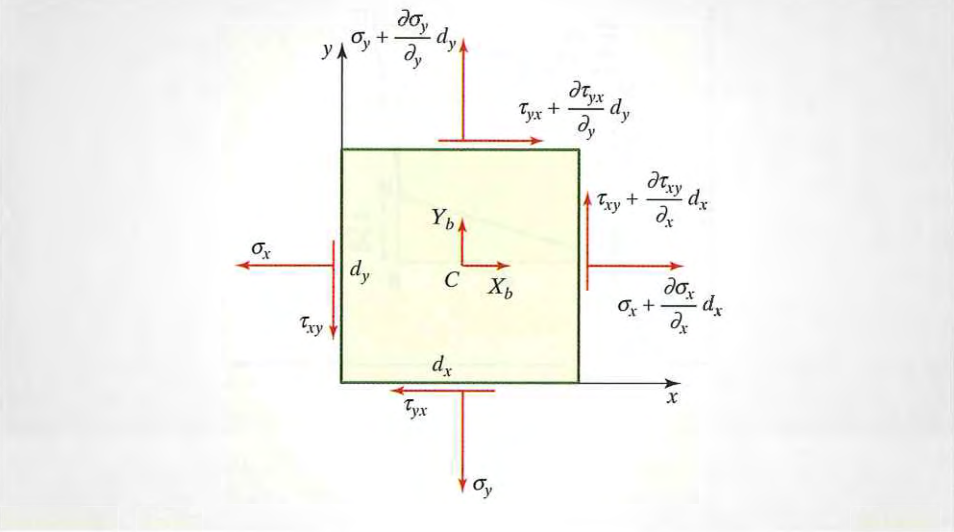
$$\gamma_{xz} = \frac{\partial u}{\partial z} + \frac{\partial w}{\partial x} \tag{1.36}$$

$$\gamma_{yz} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial y} \tag{1.37}$$

1.6 Equations of Equilibrium

For simplicity and to avoid clutter, let's consider a two-dimensional plane element of dimensions dx and dy, subjected to normal stresses σ_x and σ_y, in-plane shear stress τ_{xy}, and body forces X_b and Y_b, acting at the centroid C (Figure 1.14). The shear and normal stresses are assumed to be constants along the face on which they act but vary from one face to the opposite face.

Figure 1.14 A two-dimensional differential element in equilibrium



By resolving the forces in the x and y directions, it can be shown that

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + X_b = 0 \tag{1.38}$$

$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} + Y_b = 0 \tag{1.39}$$

Taking moment about the centroid and neglecting the high-order terms, it can be shown that $\tau_{xy} = \tau_{yx}$. In three dimensions, the above equilibrium equations become

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{xz}}{\partial z} + X_b = 0 \tag{1.40}$$

$$\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{yz}}{\partial z} + Y_b = 0 \tag{1.41}$$

$$\frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma_z}{\partial z} + Z_b = 0 \tag{1.42}$$

In addition, $\tau_{xy} = \tau_{yx}, \tau_{xz} = \tau_{zx}$, and $\tau_{yz} = \tau_{zy}$.

1.7 Laboratory Measurements and Measuring Devices

A wide range of materials such as concrete, steel, wood, and metals are being used in civil engineering projects. Their properties can vary between projects. The designer expects to know the engineering properties of these materials that are used in the project, so that sensible parameters can be used in the design and

Table 1.2

Measurements and devices

Measure	Device
Length	Ruler/scale, dial gauge, micrometer, Vernier calliper, LVDT*
Mass	Balance/scale—available in different forms (e.g., analytical)
Time	Stopwatch
Temperature	Thermometer
Load	Load cell, proving ring
Pressure	Pressure cells, pressure transducers
Strain	Strain gauge
Viscosity	Rheometer, viscometer

* *Linear variable differential transformer*

analysis. Often, the material manufacturers or suppliers carry out quality control tests from which they specify these parameters. It is also common for the users to verify these through supplementary laboratory and/or field tests. Table 1.2 lists some of the key measures and the devices used to measure them. In addition, there are nondestructive testing methods (e.g., acoustic emission, wave velocity measurements, piezoelectric devices) that can measure parameters including strength and stiffness indirectly. The nondestructive testing methods are very effective on existing structures where the properties can be determined without having the need to take any specimens from them.

It is necessary to calibrate the measurement devices on a regular basis. Here, the devices are checked against those which are known to be reliable and accurate. While some of the simple calibrations can be carried out in most laboratories, many calibrations would require sending the devices to special calibration laboratories. Figure 1.15 shows the calibration of a pressure gauge using steel weights.

Resolution of a measuring device is the smallest change it can display (e.g., 0.001 g in a digital balance). *Sensitivity* refers to the response of the instrument to a unit input (e.g., 100 millivolt per mm movement of an LVDT). *Accuracy* is how close the measurement is to the true value. *Precision* is a measure of how close the measured values are to their mean value, which may still be quite different from the true value.

The test measurements and any computed results must be reported to appropriate *significant figures*. For example, when reporting the mass of a test specimen as 245.6 g, there are four significant figures. Reporting too many digits can give a false sense of accuracy and show a lack of appreciation for the quality of the measurements. No one wants to report the strength of concrete as 31.0236 MPa, based on a uniaxial compression test. The number of significant digits should reflect the specimen size, resolution of the measuring device, and the sensitivity of the instrument. Reporting the mass of a specimen as 34.2 g is different from 34.200 g. In the latter, it is clear that the measurement is precise to 3 decimals.

Figure 1.15 Calibration of a pressure gauge

(Courtesy of N. Sivakugan, James Cook University, Australia)

1.8 Material Variability and Sample Statistics

A basic knowledge of probability and statistics is necessary when looking at the material variability and dealing with several *specimens* from the same *sample*. Note that specimen is a subset of sample. You can test tens of specimens from a sample, and there can be some scatter in the results, which have to be interpreted and presented in a meaningful way.

When a parameter (X) is measured n times, the average value is known as the *mean* ($\bar{X}$) and is given by

$$\bar{X} = \frac{1}{n}\sum_{i=1}^{n} X_i \tag{1.43}$$

Sometimes it is denoted by μ_x. *Standard deviation* of X, commonly denoted by s_x or σ_x, is a measure of the variability. It is defined as

$$s_x = \sqrt{\frac{1}{n}(X - \bar{X})^2} \tag{1.44}$$

Variance of X is simply the square of the standard deviation. A sample with $\bar{X} = 10$ and $s_x = 2$ has greater variability than the one with $\bar{X} = 20$ and $s_x = 2$. The *coefficient of variation (COV)* is a better parameter than the standard deviation to reflect the material variability because it takes the mean into account. It is expressed as a percentage and is defined as

$$COV = \frac{s_x}{\bar{X}} \times 100\% \tag{1.45}$$

1.9 Numerical Modeling

In structural, geotechnical, hydraulics, and other civil engineering areas, numerical modeling is a powerful tool used in the design offices. Here, software packages (e.g., ANSYS), based on finite element or finite difference analysis, are used to model the problem and complement the analytical methods used in the designs. Finite element and finite difference analysis are two different numerical modeling techniques. Here, the problem domain is divided into a mesh, consisting of thousands of elements and nodes. Boundary conditions and appropriate constitutive models (e.g., linear elastic, Mohr-Coulomb) are applied, and equations are developed for all the nodes. Solving the thousands of equations, the variables at the nodes are determined.

Some of these software packages and the fundamentals of numerical modeling are introduced in the later years of your degree program. In all numerical modeling, it is necessary to assume an appropriate constitutive model for the material and realistic input parameters to ensure realistic output. The quality of the output can only be as good as the quality of the input, no matter how sophisticated the software is.

Especially in situations where the boundary conditions are complex or poorly defined, numerical modeling can be the way to go. In addition, numerical models can be very effective tools for carrying out sensitivity analysis where different scenarios can be explored by varying the design parameters. They can be also effective in validating new theories and approximations used in the analytical models.

1.10 Standards, Units, and Safety

In this section, some important issues concerning material testing are discussed.

Standards

When the material is tested to determine the properties, it is necessary to follow some standard procedure to minimize the variability in the test methods and the apparatus, which can affect the results. These procedures are suggested in standards or codes, which are specific to the country. ASTM International (ASTM), International Organization for Standardisation (ISO), German Institute for Standardization (DIN), British Standards Institution (BSI), and Standards Australia (AS) are some of the organizations that develop such standards. Out of these, ASTM is the most popular and is being used worldwide.

ASTM International develops standards covering all the materials discussed in this book, including soils, rocks, aggregates, geosynthetics, asphalt, cement, concrete, metals, steel, ceramics, composites, and wood. There are more than 12,000 different standards that are published in 82 volumes that are separated, into 15 sections. The volumes can be bought on CD or print or can be accessed online. One of the sections that is of interest to civil engineers is *Section 4: Construction*, which covers many of the civil engineering materials discussed in this book.

Units

In engineering profession, the correct units must be displayed at all times in communicating the results, either verbally or in writing. An incorrect or missing unit can be misleading. Length is a physical quantity that can be expressed in the unit of meters, feet, or miles. There are several systems of units including Imperial, metric, SI, and U.S. Customary units.

The metric system was originally introduced by France in 1799, which became widely used internationally, with the exception of the United States, Myanmar, and Liberia. The International System of Units, known as SI, is the modern version of the metric system. SI units are used throughout this book. It is based on the following seven mutually independent *base units:*

- Length in meters (m)
- Mass in kilograms (kg)
- Time in seconds (s)
- Electric current in amperes (A)
- Thermodynamic temperature in kelvin (K)
- Luminous intensity in candelas (cd)
- Amount of substance in moles (mol)

Only the first three of these are commonly used in civil engineering. The units for all other physical quantities are derived from any of these seven base units. Prefixes indicate orders of magnitude in steps of 1000 and provide a convenient way to express small and large numbers. The prefixes used with SI units are summarized in Table 1.3.

Some of the derived quantities have special names. For example:

- newton (N) = Force required to accelerate a 1.0 kg mass as the rate of 1.0 m/s^2
- pascal (Pa) = N/m^2
- joule (J) = Work done when a force of 1.0 newton moves by a distance of 1.0 m
- watt = joule/s

Other special names that are not necessarily SI that are still worth noting are:

- angstrom (Å) = 10^{-10} m
- bar = 10^5 Pa
- British (short) ton = 2000 lb
- British (long) ton = 2240 lb

SI Prefixes used with units Table 1.3

Factor	Prefix	Symbol
10^{24}	yotta	Y
10^{21}	zeta	Z
10^{18}	exa	E
10^{15}	peta	P
10^{12}	tera	T
10^{9}	giga	G
10^{6}	mega	M
10^{3}	kilo	k
10^{2}	hecto	h
10^{1}	deca	da
10^{-1}	deci	d
10^{-2}	centi	c
10^{-3}	milli	m
10^{-6}	micro	?
10^{-9}	nano	n
10^{-12}	pico	p
10^{-15}	femto	f
10^{-18}	atto	a
10^{-21}	zepto	z
10^{-18}	yocto	y

- dyne = 10^{-5} N
- hectare (ha) = 10000 m^2
- hertz = 1.00 cycle/s
- hundred weight (UK) = 112 lb
- hundred weight (US) = 100 lb
- kilopond = kilogram-force
- kip = 1000 lb-force (lbf)
- metric ton (tonne) = 1000 kg
- nautical mile = 1852 m
- poundal = force required to accelerate a mass of 1.0 lb at the rate of 1.0 ft/s^2
- stone (US) = 12.5 lb; stone (UK) = 14.0 lb

Some notes from ASTM that should be observed when expressing SI units are as follows.

- In derived units formed by multiplication, use *raised* dots (·) between the symbols. For example; N·m, Pa·s, etc. The unit names should be written as newton meter, pascal second and so forth, with a space between the different unit names.

- In derived units formed by division, use only one solidus (/) per expression and parentheses to avoid ambiguity.
- SI symbols are not abbreviations, and hence no period should follow them except at end of a sentence.
- A unit is the same whether singular or plural.
- Leave a space between the value and the symbol. For example, 6 m, not 6m. An exception is for the plane angle degree which is expressed as, for example, 31° and not 31 °. The symbol for degree Celsius is °C.
- Do not place a space or hyphen between the prefix and the unit name. For example, kilogram, not kilo-gram.
- Choose a prefix so that the number lies between 0.1 and 1000. For example 3.2×10^{-4} cm/s.

Some unit conversions that are useful in civil engineering are given in Appendix A.

Safety

Workplace health and safety play a major role in all construction sites and material testing laboratories. The importance and the emphasis on safety are increasing every year, with little or no tolerance for any workplace-related accidents or injuries. Laboratories with machinery, equipment, and hazardous materials are some of the high-risk areas that are prone to accidents. MSDS (Material Safety Data Sheets) must be used to assess whether special protective clothing and/or eye protection are required. Hazardous chemicals should be disposed of in an appropriate manner. Every possible step must be taken to minimize the risk. Designated workplace health and safety officers give induction for the beginners and ensure that those working in the laboratories follow the best practices as far as the workplace health and safety is concerned.

1.11 Sustainability

Sustainability is an important consideration these days in all engineering disciplines. At the university level, it is expected that sustainability concepts are embedded in every subject. It is also expected that sustainability is built into all engineering projects. The American Society of Civil Engineers (ASCE) defines sustainability as a set of economic, environmental, and social conditions in which all of society has the capacity and opportunity to maintain and improve its quality of life indefinitely without degrading the quantity, quality, or availability of natural, economic, and social resources.

Society has entrusted civil engineers to create a sustainable world through responsible material selection, design methods, and construction practices. They act as a bridge between science and society and play a leading role in planning, designing, building, and ensuring a sustainable future. They often work in multidisciplinary teams with other professionals such as geologists, ecologists, economists, sociologists, and others to effectively address the challenges of sustainable development. Chapter 13 discusses the sustainable use of civil engineering materials.

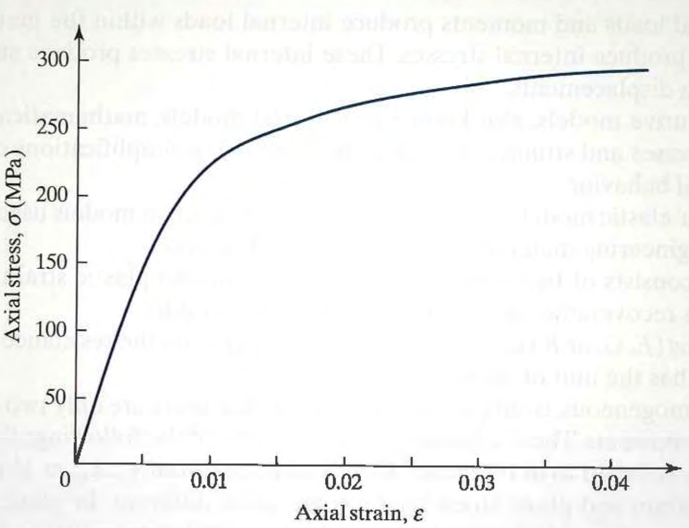
1.12 Summary

1. External loads and moments produce internal loads within the material, which in turn produce internal stresses. These internal stresses produce strains, which result in displacements.
2. Constitutive models, also known as material models, mathematically describe how stresses and strains are related. They are often simplifications of the actual material behavior.
3. A linear elastic model is one of the popular constitutive models used to describe civil engineering materials, especially at small strains.
4. Strain consists of two components: elastic strain and plastic strain. The elastic strain is recoverable, and plastic strain is irrecoverable.
5. Modulus (E, G, or K) is a measure of stiffness, which is the resistance to deformation. It has the unit of stress.
6. In a homogeneous, isotropic, and elastic material, there are only two *independent* elastic constants. These constants can be any two of the following: E, ν, G, and K.
7. What is referred to in literature as ε_{xy} is different from γ_{xy}. $\varepsilon_{xy} = \frac{1}{2}\gamma_{xy}$.
8. Plane strain and plane stress loadings are quite different. In plane strain loading, there still can be normal stresses (not strains) perpendicular to the plane. Similarly, in plane stress loading, there can be normal strains (not stresses) perpendicular to the plane.
9. The laboratory measurement devices should be routinely calibrated to ensure that they give the correct readings. The best practices in workplace health and safety should be followed in the material testing laboratories and construction sites.
10. Correct dimensions and units should be used in any oral or written communications concerning designs or material testing.

Exercises

1. Which of the following materials is least used in roadwork?
   - **a.** Soils
   - **b.** Rocks
   - **c.** Asphalt
   - **d.** Steel
2. Which of the following materials has the largest Young's modulus?
   - **a.** Rocks
   - **b.** Concrete
   - **c.** Wood
   - **d.** Plastics
3. Which of the following values is unlikely for the Poisson's ratio of an engineering material?
   - **a.** 0.1
   - **b.** 0.3
   - **c.** 0.5
   - **d.** 0.7

Figure 1.16 Stress-strain diagram for Exercise 3



4. Which of the following is true about an isotropic linear elastic material?

- **a.** The material shows the same properties in all directions.
- **b.** When subjected to external loads, the stress is proportional to strain.
- **c.** The Young's modulus is the same at any stress.
- **d.** All the above are true.

5. Which of the following is not reported to four significant figures?

- **a.** 245.0
- **b.** 5.230×10^2
- **c.** 0.0014
- **d.** 34.20

6. Which of the following is not a unit for mass?

- **a.** kg
- **b.** kN
- **c.** tonne
- **d.** pound

7. Which of the following is the smallest?

- **a.** micrometer
- **b.** nanometer
- **c.** angstrom
- **d.** millimeter

8. What is the difference between an elasto-plastic model and an elastic-plastic model?

9. The stress-strain relationship of a nonlinear hyperbolic model is given by

$$\sigma = \frac{\varepsilon}{a + b\varepsilon} \tag{1.7}$$

Show that the slope of the σ-ε plot at zero strain and the ultimate stress are given by $1/a$ and $1/b$, respectively.

10. The stress-strain diagram developed by subjecting a metal bar to a tensile load is shown in Figure 1.16. Estimate the yield stress and the Young's modulus. What would be the plastic strain when the longitudinal strain is 3%?

11. A computer-generated output shows the following stresses at a point: $\sigma_{xx} = 40$ MPa, $\sigma_{yy} = 20$ MPa, $\sigma_{zz} = -10$ MPa, $\tau_{xy} = 15$ MPa, $\tau_{yz} = -5$ MPa and $\tau_{zx} = -25$ MPa. Show these stresses, with arrows in the appropriate directions, in a three-dimensional element.

12. From Eqs. 1.15 and 1.16, deduce the expressions for the stresses and strains in three-dimensional space when the stresses acting on the element are principal stresses.

13. Surf the web and discuss the different systems of units and the advantages of SI over the others.

14. Write a short essay of five hundred words on nondestructive testing in civil engineering.

15. Do a literature review and discuss the use of the following in civil engineering: impact echo tester, half-cell potential meter, acoustic emission, scanning electron microscope.

16. Surf the web and list some major software packages used for modeling structural and geotechnical problems. Discuss their key features and limitations.

17. Express the following in SI units:

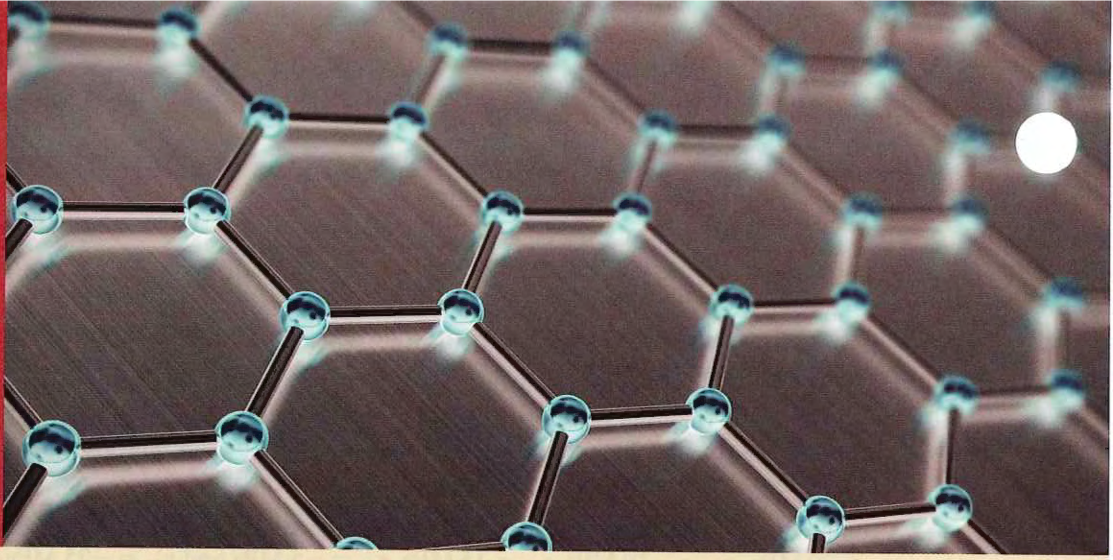
a. angstrom
b. kip
c. psi
d. bar

18. How many significant figures are there in the following numbers?

a. 254 million
b. 0.00034
c. 0.0023500
d. 4.7690
e. 4.23×10^{-2}

2

CHAPTER



The properties of materials are dictated by their chemical structures. Graphene is a crystalline single atomic layer of carbon in the form of graphite, with extraordinary properties. Rost9/Shutterstock.com

Chemistry of Materials

2.1 Introduction

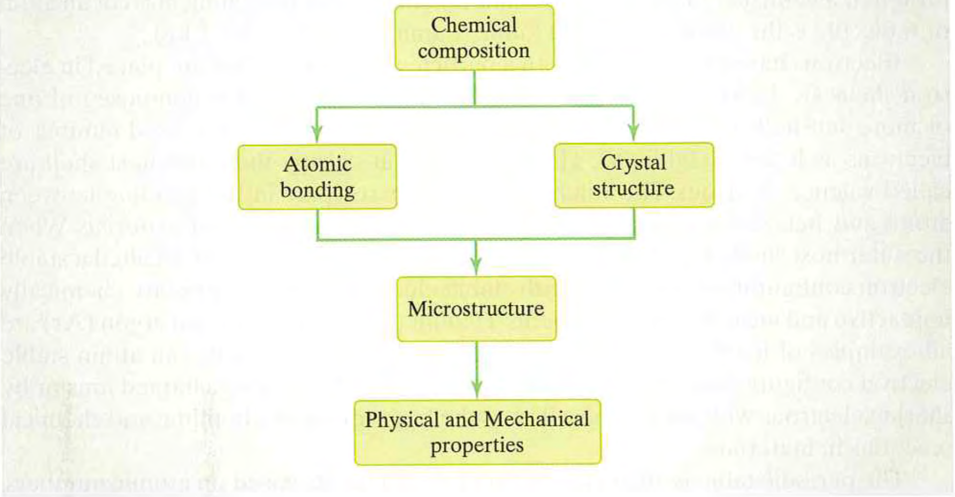
Civil engineers are often involved in design structures, including buildings, bridges, and tunnels, which require use of different types of engineering materials such as metals, ceramics, composites, and polymers. As illustrated in Figure 2.1, the physical and mechanical properties of materials primarily depend on their microstructures, which are dictated by the chemical composition, atomic bonding, and crystal structure of the materials. Also, the arrangement of atoms and imperfections in the materials influence their mechanical properties. Hence, it is important that civil engineers understand these fundamental properties to appreciate the advancements in materials science and engineering over the years that have paved the way for developing new materials for engineering applications. Generally speaking, materials science and engineering is an integral part of civil engineering. In fact, to effectively create new civil structures, high-performance materials that will lead to less materials consumption need to be developed. This chapter introduces the fundamental chemical aspects of materials at atomic level, including atomic structure, atomic bonding, crystal structures, imperfections, and strengthening mechanisms of materials. In addition, a broad classification of materials and their applications and the basic characterization techniques are also presented.

2.2 Atomic Structure and Bonding

Understanding of the atomic structure and bonding of a material is important for engineers, because they have a profound influence on a material's physical, chemical and mechanical properties. Importantly, the properties of a material can be tailored for specific applications by modifying the microstructure of the material.

Correlation of the chemical nature of materials and their physical and mechanical properties

Figure 2.1

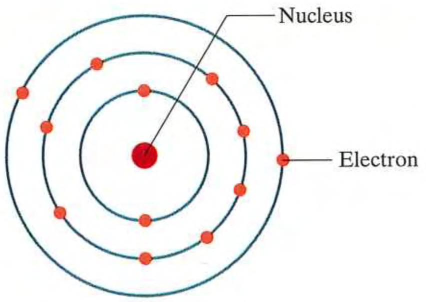


2.2.1 Atomic Structure

An atom is composed of a nucleus that contains neutrally charged neutrons and positively charged protons, with negatively charged electrons around the nucleus (Figure 2.2). The protons and neutrons in the nucleus are held together by a strong nuclear force, and the positively charged nucleus and the negatively charged electrons are attracted by coulombic attraction. In a natural state, the number of protons and electrons are equal, and hence the net charge of an atom is zero. The atomic number of an element is the number of electrons or protons in the atom. For example, the atomic number of magnesium (Mg) metal is 12 because it contains 12 electrons and 12 protons. The atomic mass of an element is the number of protons and neutrons in

Electronic configuration of magnesium (Mg) atom, which contains 12 protons and 12 neutrons in the nucleus and 12 electrons around the nucleus

Figure 2.2



the atom, so for magnesium metal, there are 12 protons and 12 neutrons, which make the atomic mass of magnesium 24. The weight of an electron is not included in the atomic mass calculation, since its weight is orders of magnitude lower than that of a proton or a neutron (Table 2.1). The standard unit used for indicating mass of an atom or molecule is the atomic mass unit (amu; $1 \text{ amu} = 1.6605 \times 10^{-27}$ kg).

Electrons have properties of both a particle and a wave. They are placed in electron shells (K, L, M, N ...) around the nucleus, and each shell is composed of one or more subshells (s, p, d, f ...). The shells and subshells contain a fixed number of electrons as listed in Table 2.2. The electrons that occupy the outermost shell are called valence electrons. The valence electrons participate in the bonding between atoms and, hence, dictate many physical and chemical properties of materials. When the outermost shell of an atom is completely filled with electrons, it is called a stable electron configuration. Elements with stable electron configurations are chemically unreactive and are called inert elements. Helium (He), neon (Ne), and argon (Ar) are all examples of inert elements. Atoms with unfilled valence shells can attain stable electron configurations by losing or gaining electrons by forming charged ions or by sharing electrons with other atoms. This is the basis for atomic bonding and chemical reactions in materials.

The periodic table is an arrangement of the elements based on atomic numbers, electron configurations, and chemical properties (Figure 2.3). The periodic table can be classified into 4 blocks: s-block, d-block, p-block, and f-block. The s-block contains

Table 2.1 Properties of particles in an atom

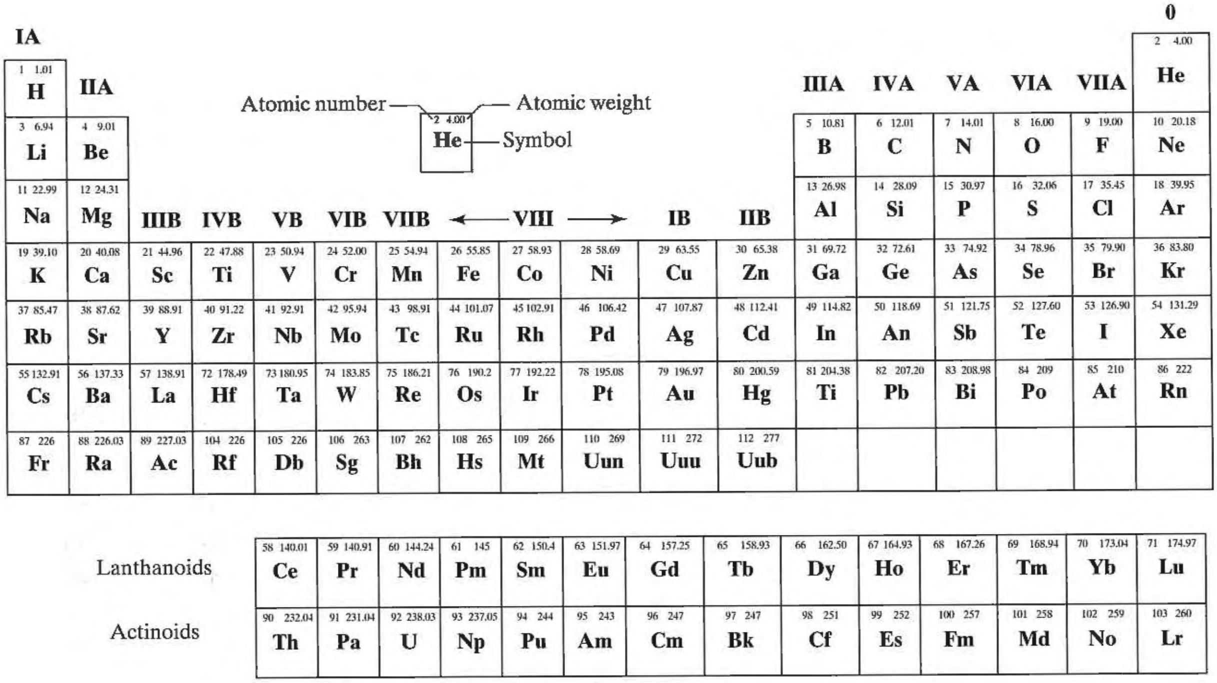
Particle	Charge	Mass (kg)
Proton	Positive	1.6726×10^{-27}
Neutron	Neutral	1.6749×10^{-27}
Electron	Negative	9.1094×10^{-32}

Table 2.2 Maximum number of electrons present in the shells and subshells

Shell Designation	Subshells	Number of Electrons	
		Subshell	Shell
K	s	2	2
L	s	2	8
	p	6	
M	s	2	18
	p	6	
	d	10	
N	s	2	32
	p	6	
	d	10	
	f	14	

Periodic table of the elements

Figure 2.3



IA	IIA	IIIB	IVB	VB	VIB	VIIB	VIII	VIII	VIII	IB	IIB	IIIA	IVA	VA	VIA	VIIA	0
1 1.01 H																	2 4.00 He
3 6.94 Li	4 9.01 Be											5 10.81 B	6 12.01 C	7 14.01 N	8 16.00 O	9 19.00 F	10 20.18 Ne
11 22.99 Na	12 24.31 Mg											13 26.98 Al	14 28.09 Si	15 30.97 P	16 32.06 S	17 35.45 Cl	18 39.95 Ar
19 39.10 K	20 40.08 Ca	21 44.96 Sc	22 47.88 Ti	23 50.94 V	24 52.00 Cr	25 54.94 Mn	26 55.85 Fe	27 58.93 Co	28 58.69 Ni	29 63.55 Cu	30 65.38 Zn	31 69.72 Ga	32 72.61 Ge	33 74.92 As	34 78.96 Se	35 79.90 Br	36 83.80 Kr
37 85.47 Rb	38 87.62 Sr	39 88.91 Y	40 91.22 Zr	41 92.91 Nb	42 95.94 Mo	43 98.91 Tc	44 101.07 Ru	45 102.91 Rh	46 106.42 Pd	47 107.87 Ag	48 112.41 Cd	49 114.82 In	50 118.69 An	51 121.75 Sb	52 127.60 Te	53 126.90 I	54 131.29 Xe
55 132.91 Cs	56 137.33 Ba	57 138.91 La	72 178.49 Hf	73 180.95 Ta	74 183.85 W	75 186.21 Re	76 190.2 Os	77 192.22 Ir	78 195.08 Pt	79 196.97 Au	80 200.59 Hg	81 204.38 Ti	82 207.20 Pb	83 208.98 Bi	84 209 Po	85 210 At	86 222 Rn
87 226 Fr	88 226.03 Ra	89 227.03 Ac	104 226 Rf	105 226 Db	106 263 Sg	107 262 Bh	108 265 Hs	109 266 Mt	110 269 Uun	111 272 Uuu	112 277 Uub						

Lanthanoids	58 140.01 Ce	59 140.91 Pr	60 144.24 Nd	61 145 Pm	62 150.4 Sm	63 151.97 Eu	64 157.25 Gd	65 158.93 Tb	66 162.50 Dy	67 164.93 Ho	68 167.26 Er	69 168.94 Tm	70 173.04 Yb	71 174.97 Lu
Actinoids	90 232.04 Th	91 231.04 Pa	92 238.03 U	93 237.05 Np	94 244 Pu	95 243 Am	96 247 Cm	97 247 Bk	98 251 Cf	99 252 Es	100 257 Fm	101 258 Md	102 259 No	103 260 Lr

alkali metals and alkaline earth metals (IA and IIA) as well as hydrogen and helium. The d-block contains the transition metals (IIIB to IIB). The p-block contains elements in group IIIA to VIIA and the metalloids. The lanthanides and actinides belong to the f-block.

2.2.2 Bonding

There are three types of bonds in solid materials: metallic bonds, covalent bonds, and ionic bonds. These bonds are relatively strong and are called primary or chemical bonds. In many solid materials, physical forces or weak bonding also exists. This type of bonding is known as secondary bonding or van der Waals bonding. The strength of the bonds depends on the binding energies. Typical binding energies for the four types of bonds are listed in Table 2.3. Materials with high binding energy generally possess high strength and high melting points.

Binding energies for the four types of bonds

Table 2.3

Bond	Binding Energy (kcal/mol)
Ionic	150–370
Covalent	125–300
Metallic	25–200
van der Waals	<10

Metallic Bond

A metallic bond is the electrostatic attractive force between the delocalized/valence electrons and the positively charged metal ions and is found in metals and their alloys; it forms a close-packed arrangement in these materials (Figure 2.4). Metallic elements such as iron, aluminium, and magnesium donate their valence electrons to form a "sea" of electrons surrounding the atoms. For example, magnesium donates two valence electrons and acquires a positive charge of two. The valence electrons move freely within the electron sea, and the positively charged ion cores are held together by mutual attraction to the electrons, and hence a strong metallic bond is formed. Metals are good conductors of electricity due to their valence electrons. When a voltage is applied to a metal, the valence electrons of the metal move, causing a current to flow. Since metallic bonding is strong, metals exhibit high Young's modulus and relatively high melting points. Metallic bonds are nondirectional, meaning that the magnitude of bond is equal in all directions. Metals are not only good conductors of heat, which occurs by vibration of the positive ions and the valence electrons, but also are malleable and ductile, making metals suitable for forming into different shapes.

Covalent Bond

Covalent bonding in materials is formed by sharing of valence electrons among two or more atoms. Nonmetals readily form covalent bond with other nonmetals to achieve a stable electron shell configuration. For example, a carbon atom shares its four valance electrons with four hydrogen atoms (each of which has one valence electron) to form methane (CH_4) (Figure 2.5). Covalent bonds are found in many

Figure 2.4 Metallic bonding in metals

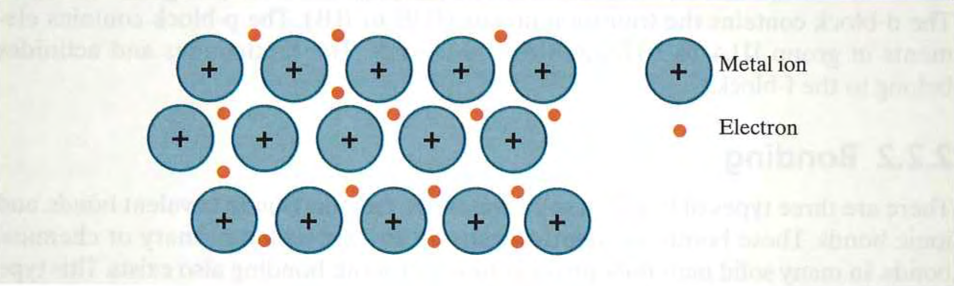
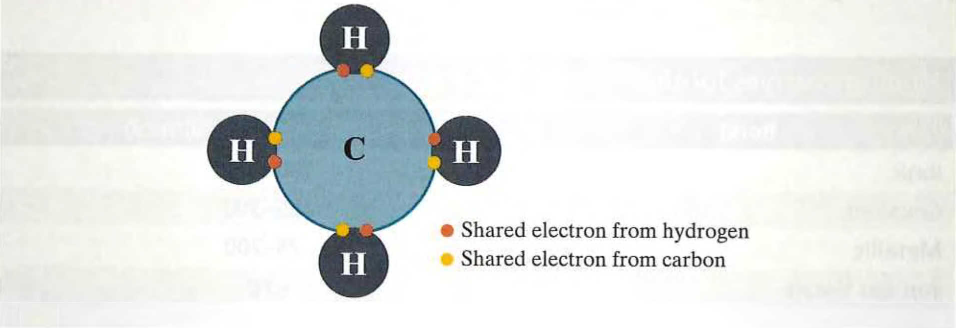


Figure 2.5 Covalent bonding in methane (CH_4), an organic compound.



polymers (for example, polyethylene and polypropylene) and ceramics (for example, silicon carbide and boron nitride), and also in gas molecules such as hydrogen (H_2), oxygen (O_2), and nitrogen (N_2). Most inorganic materials with covalent bonds exhibit high strength and hardness and have high melting points. However, the ductility of a covalently bonded material is not high as that of a metallic-bonded material, such as metals and alloys, since covalent bond is directional. Also, unlike metallic bonds, the valence electrons in a covalent bond are interlocked; hence, covalently bonded materials are poor electrical conductors.

Ionic Bond

Ionic bonding is formed between charged atoms where an electron is donated from one electropositive atom to an electronegative atom, and as a result, a positively charged cation and negatively charged anion are produced. Ionic bonding is a result of an electrostatic attraction between two oppositely charged ions. Generally, the cation is a metal atom, and the anion is a nonmetal atom. For example, in NaCl (common salt), a sodium (Na) atom loses an electron and a chlorine (Cl) atom gains that electron, thus forming Na^+ and Cl^- ions, respectively. The oppositely charged ions are attracted to form sodium chloride (NaCl) (Figure 2.6). Ionic bonding is commonly found in many ceramics. These materials generally possess high mechanical strength and hardness but very low ductility. Ionically bonded materials have relatively high melting points. These materials have poor electrical conductivity in solid form, but in molten form they conduct electricity by free movement of ions.

van der Waals bonding

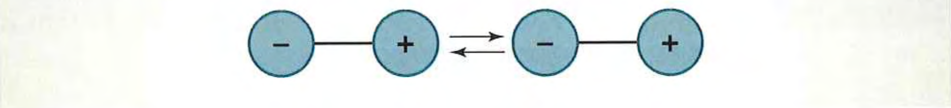
A van der Waals bond, also known as a secondary bond, is a weak bond as compared to primary bonds. In this bonding, the attractive forces arise from atomic or molecular dipoles (Figure 2.7). This bonding occurs in molecules where the atoms are already

Ionic bonding in common salt, sodium chloride (NaCl)

Figure 2.6

van der Waals bonding between two dipoles

Figure 2.7



strongly bonded by covalent or ionic bonding. Inert gases such as argon and neon exhibit secondary bonding. In fact, properties of liquids such as surface tension, boiling points, and solubilities are determined by this type of bonding.

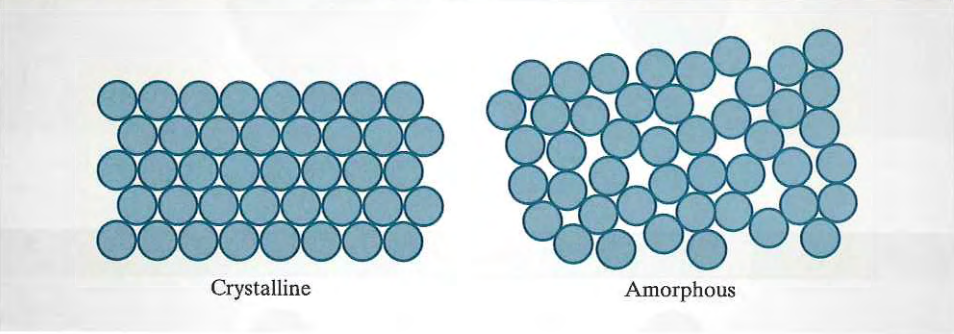
Hydrogen bonding, a type of van der Waals bonding, is found in some molecules that have hydrogen as one of the elements. For example, water (H_2O), ammonia (NH_3), and organic molecules, such as DNA, exhibit hydrogen bonding. Hydrogen bonding can be intermolecular (between molecules) or intramolecular (within a single molecule). It should be noted that the atoms in H_2O are already covalently bonded, but hydrogen bonding occurs between H_2O molecules. In fact, the reduction in density of water when frozen can be explained by the change in the structure of water due to intermolecular hydrogen bonding, which is also responsible for the high boiling point of water (100 °C).

2.3 Arrangement of Atoms

Based on the arrangement of atoms, solid materials can be broadly classified into two groups: crystalline materials and noncrystalline materials (Figure 2.8). Generally, the properties of materials are directly related to their crystal structures. If the atoms in a material are placed in a repeated fashion over large atomic distances, then the material is called crystalline. Almost all metals and many ceramics have crystalline structure. Noncrystalline materials do not have a long-range atomic order. These materials are called *amorphous* materials. Glasses and many polymers are amorphous in nature.

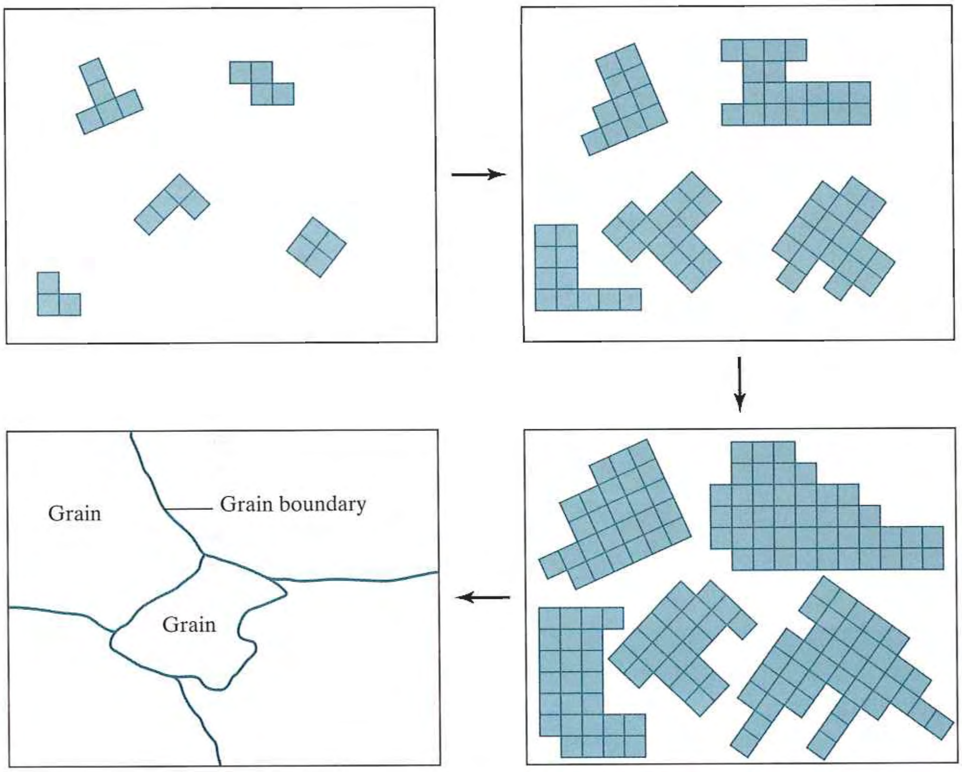
Polycrystalline materials contain grains and grain boundaries. Grains are a collection of many small crystals, and the mismatched region between two grains is called the grain boundary. The stages involved in the formation of grains and grain boundaries during the solidification process (liquid melt to solid metal) are illustrated in Figure 2.9. The first stage is the formation of nuclei (small crystals) in the liquid melt, followed by the random growth of crystals. The crystal growth is stopped when the crystals grow larger and impinge with the neighboring crystals. There will be a mismatch in atomic arrangement in the region between the grains, and this region is called the *grain boundary*.

Figure 2.8 Arrangements of atoms in crystalline and amorphous materials



Stages involved during the solidification of metal, forming grains and grain boundaries.

Figure 2.9



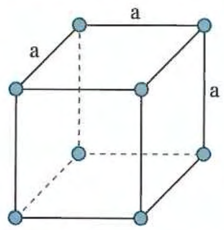
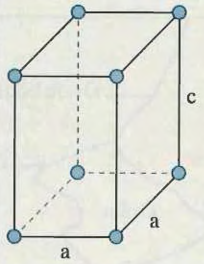
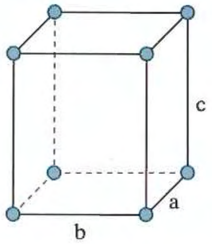
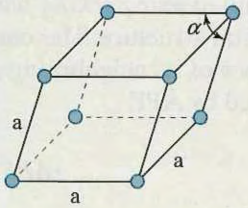
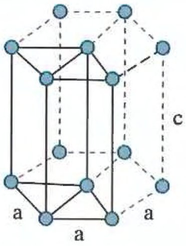
(Adapted from Callister W. D., and Rethwisch D.G. 2010)

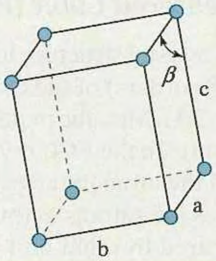
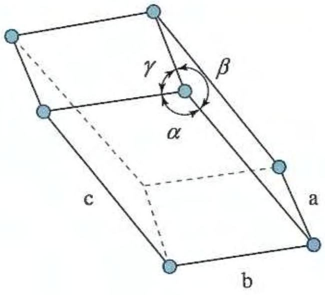
The basic unit of the crystal structure is called the unit cell. The coordination number and *atomic packing factor* (APF) are the two other important characteristic of a crystal structure. The coordination number of a central atom of a crystal is the number of its neighboring atoms. The closeness of the atoms present in a crystal is defined by APF.

$$APF = \frac{volume\ of\ atoms\ in\ a\ unit\ cell}{total\ unit\ cell\ volume} \tag{2.1}$$

A large single crystal can be formed when the repeated arrangement of atoms is present throughout the specimen without any mismatch. Single crystals can be produced artificially and have wide applications in the semiconductor industry. Metallic materials are generally polycrystalline, meaning that they are composed of many small crystals, otherwise known as grains. There are basically seven crystal systems: cubic, tetragonal, orthorhombic, rhombohedral, hexagonal, monoclinic, and triclinic (Table 2.4). But, a number of crystal structures are known to exist, which could be described as patterns formed by repeated crystal systems. However, in this section we will only discuss three crystal structures (body-centered cubic, face-centered cubic, and hexagonal close-packed) that are mostly found in metals.

Table 2.4 The seven basic crystal systems and lattice parameter

Crystal System	Axial Relationship; Interaxial Angles	Unit Cell Geometry
Cubic	$a = b = c$; $\alpha = \beta = \gamma = 90°$	
Tetragonal	$a = b \neq c$; $\alpha = \beta = \gamma = 90°$	
Orthorhombic	$a \neq b \neq c$; $\alpha = \beta = \gamma = 90°$	
Rhombohedral (Trigonal)	$a = b = c$; $\alpha = \beta = \gamma \neq 90°$	
Hexagonal	$a = b \neq c$; $\alpha = \beta = 90°, \gamma \neq 120°$	

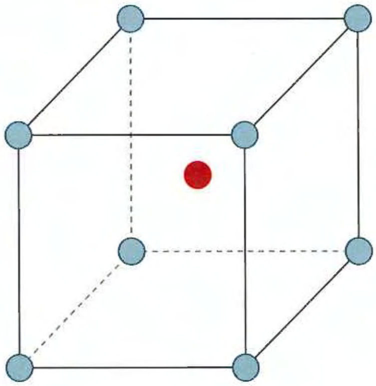
Monoclinic	$a \neq b \neq c$; $\alpha = \gamma = 90° \neq \beta$	
Triclinic	$a \neq b \neq c$; $\alpha \neq \beta \neq \gamma \neq 90°$	

Body-Centered Cubic (BCC)

A BCC crystal structure has a cubic unit cell with atoms placed at all eight corners and a single atom at the center of the cube (Figure 2.10). Metallic elements such as iron, chromium, and tungsten have a BCC structure. In a single BCC unit cell, there are two atoms in total, which is one atom from the eight corners (eight corner atoms shared by eight unit cells; 8 atoms × 1/8 = 1 atom) and one atom from the center of the unit cell. The BCC crystal structure has a coordination number of 8, meaning the corner atoms and the center atoms have eight neighbor atoms. The atomic factor for BCC is 0.68.

Body-centered cubic (BCC) crystal structure

Figure 2.10



Face-Centered Cubic (FCC)

The FCC crystal structure has a unit cell of cubic geometry with atoms located at each corner (8 corners) of the cube and the centers of all the cube faces (6 faces) as shown in Figure 2.11. Metallic materials such as aluminium, copper, and silver have FCC crystal structure. In the FCC crystal structure, each corner atom is shared within eight unit cells, and the atom in the center of each face is shared by two unit cells. A single FCC crystal has four atoms in total, which is one atom from the eight corners (eight corner atoms shared by eight unit cells; 8 atoms × 1/8 = 1 atom) and three atoms from the six faces (each atom in the six faces is shared by 2 unit cells; 6 atoms × 1/2 = 3 atoms). The FCC crystal structure has a coordination number of 12 such that 6 neighbor atoms are in the same close-packed plane, 3 atoms are in the layer above, and 3 atoms are in the layer below. The atomic packing factor for FCC is 0.74.

Hexagonal Close-Packed (HCP)

The HCP crystal structure is slightly more complex than FCC and BCC crystal structures. As shown in the Figure 2.12, the top and bottom hexagonal-shaped faces of the unit cell consist of six atoms at each corner and one atom in the center.

Figure 2.11 Face-centered cubic (FCC) crystal structure

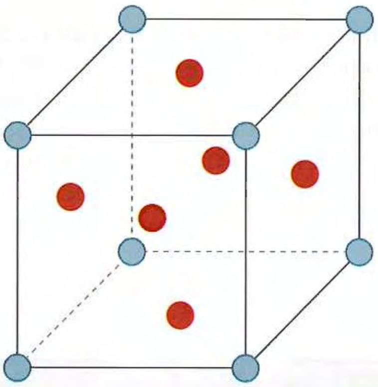
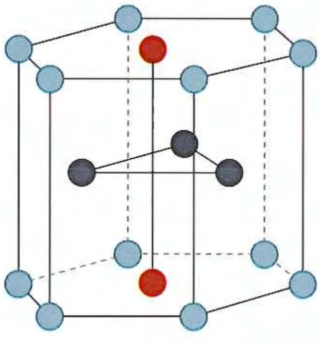


Figure 2.12 Hexagonal close-packed (HCP) crystal structure



Another plane of three atoms is placed between the top and the bottom planes. Metals such as magnesium, zinc, and titanium have HCP crystal structure. For HCP crystal structure, a unit cell contains one atom each from the six corners of the two hexagons ($2 \times (6 \text{ atoms} \times 1/6) = 2$ atoms), one atom from the two hexagon faces ($2 \times (1 \text{ atom} \times 1/2) = 1$ atom), and three atoms in the center for a total of six atoms. The coordination number for the HCP crystal structure is 12, and the APF is 0.74.

The theoretical density of a metal can be calculated using its crystal structure, atomic radius, and atomic weight (Table 2.5). The following relationship is used to calculate the theoretical density (ρ g/cm^3) of a metal:

$$\rho = \frac{nA}{V_c N_A} \tag{2.2}$$

n = number of atoms in the unit cell
A = atomic weight (g/mol)
V_c = volume of the unit cell (cm^3)
N_A = Avogadro's number (6.022×10^{23} atoms/mol)

Table 2.5

Crystal structure and physical properties of a few popular metals

Metal	Crystal Structure	Atomic Radius (nm)	Atomic Weight (g/mol)
Iron	BCC	0.1241	55.8
Copper	FCC	0.1278	63.5
Aluminium	FCC	0.1431	26.9
Chromium	BCC	0.1249	51.9
Cobalt	HCP	0.1253	58.9
Gold	FCC	0.1442	196.9

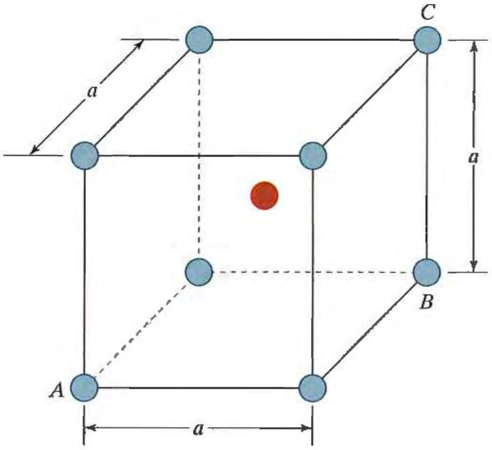
EXAMPLE 2.1

Calculate the theoretical density of iron, which has a BCC crystal structure.

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

$n = 2$
A = atomic weight of iron, 55.8 g/mol
N_A = Avogadro's number (6.022×10^{23} atoms/mol)
V_c = volume of the unit cell
R = radius of an iron atom, 0.1241 nm

Volume of a BCC Unit Cell (assume the center atom touches the corner atoms)



$$(AB)^2 = a^2 + a^2$$

$$(AC)^2 = (BC)^2 + (AB)^2$$

$$(4R)^2 = a^2 + a^2$$

$$(4R)^2=a^2+a^2$$

$$a = 4R\frac{4R}{\sqrt{3}}$$

$$V_c = a^3 = \left(\frac{4\mathrm{R}}{\sqrt{3}}\right)^3$$

$$\rho = \frac{2 \times 55.8}{\left(\frac{4\mathrm{R}}{\sqrt{3}}\right)^3 \times 6.023 \times 10^{23}}$$

$$\rho = 7.83\frac{g}{cm^3}$$

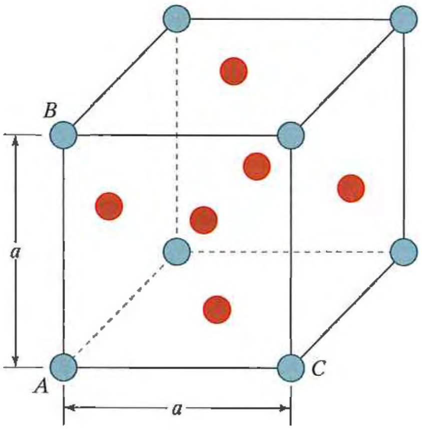
EXAMPLE 2.2

Calculate the theoretical density of copper, which has an FCC crystal structure.

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

$n = 4$
A = atomic weight of copper, 63.5 g/mol
N_A = Avogadro's number (6.022×10^{23} atoms/mol)
V_c = volume of the unit cell
R = radius of an copper atom, 0.1278 nm

Volume of an FCC unit cell (assume the atoms touch one another across a face diagonal)



$$(AB)^2 + (AC)^2 = (BC)^2$$

$$a^2 + a^2 = (4R)^2$$

$$a = 2R\sqrt{2}$$

$$V_c = a^3 = (2R\sqrt{2})^3 = 16R^3\sqrt{2}$$

$$\rho = \frac{4 \times 63.5}{\left(\frac{4R}{\sqrt{3}}\right)^3 \times 6.023 \times 10^{23}}$$

$$\rho = 8.89\ g/cm^3$$

EXAMPLE 2.3

Calculate the radius of a nickel atom. Nickel has an FCC crystal structure, a density of 8.908 g/cm^3, and an atomic weight of 58.69 g/mol.

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

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

For FCC crystal structure

$$V_c = 16R^3\sqrt{2}$$

$$R = \left(\frac{nA\sqrt{2}}{16\rho N_A}\right)^{\frac{1}{3}}$$

(Continued)

$$R = \left(\frac{4 \times 58.69 \times \sqrt{2}}{16 \times 8.908 \times 6.023 \times 10^{23}} \right)^{\frac{1}{3}}$$

$$R = 1.24 \times 10^{-8}\, cm = 0.124\, nm$$

2.4 Classification of Materials

Materials are broadly classified, based on chemical makeup and atomic structure, into three basic categories: metals, ceramics, and polymers. In addition to the three basic types of materials, composites and nanomaterials are also discussed in this section due to their importance in civil engineering applications.

Metals

Metals are generally made up of one or more metallic elements such as iron, copper, aluminum, and magnesium. A small amount of nonmetallic elements such as carbon, nitrogen, and oxygen may also be present. The product of a combination of metallic elements and/or presence of a small amount of nonmetallic elements is called an alloy. Since all metals are mostly crystalline, the atoms in metals are arranged in an orderly fashion, and hence, the density of metals is higher as compared to the amorphous ceramics and polymers. Due to metallic bonding, metals and alloys are relatively strong and also possess high ductility. Metals are good conductors of electricity and heat. Some of the metals, such as iron and nickel, have magnetic properties. Steel, an alloy of iron, is widely used in structural frameworks for buildings due to its excellent mechanical properties. However, corrosion in certain metals, including steels, is a serious problem that affects their longevity and widespread applications. Aluminum alloys, which are lightweight materials, have better corrosion resistance than steels and are finding applications in relatively low load-bearing civil structures such as window frames.

Ceramics

Ceramics are generally composed of metallic and nonmetallic elements like oxides (Al_2O_3), carbides (SiC), and nitrides (Si_3N_4) and include glass, cement, and clay. The strength, hardness, and stiffness of ceramics are high, but they are very brittle (poor ductility). These materials are widely used as insulating materials due to their high resistance to heat. They also possess high resistance to electricity. In modern buildings, the use of glass as "curtain walls" has been steadily increasing. Corrosion is not a problem with ceramics, so they are widely used as coating materials on metals to withstand aggressive corrosive environments.

Polymers

Polymers are long-chain molecules, and they mainly contain elements such as carbon (considered as the backbone for polymers), hydrogen, oxygen, and nitrogen. Polymers can be natural or synthetic (for example, polyvinyl chloride (PVC), polyethylene, nylon, and polycarbonate). In the past few decades, the use of polymers in engineering applications has grown rapidly due to their attractive properties, such as their light weight and easy fabrication. However, their mechanical strength is inferior to metals and ceramics. They also have low melting points, which make them unsuitable in high-temperature applications. Polymers, commonly known as plastics, are commercially available in different forms, such as fibers, foams, and sheets, which have engineering applications. Broadly, polymers can be classified into two groups, that is, thermoplastic and thermosetting polymers. This classification is primarily based on their response to heat. Thermoplastic polymers (for example, polyethylene) will turn soft when heated but regain their properties when cooled. The high-temperature deformation in thermoplastic polymers is due to molecular sliding. In the case of thermosetting polymers like synthetic rubbers, such as neoprene (polychloroprene)— used in electrical insulation, and Viton (fluoropolymer)— commonly used in O-rings, the materials become hard and rigid upon heating. This process is irreversible, which is due to chemical reactions involved during the heating process.

Composites

Composite materials are made up of two or more of the basic materials (metals, ceramics, and polymers). The purpose of making composite materials is to combine the attractive properties of the basic materials. For example, fiberglass and carbon fiber-reinforced polymer (CFRP) are composite materials, where fiberglass is composed of glass fibers (ceramic) and polymers, and CFRP is made of carbon fibers and polymers. These materials have low densities and exhibit high strength and ductility. Basically, there are two types of composites: metal-matrix and fiber-reinforced. Metal-matrix composites are an incorporation of ceramics in a metal matrix, for example, alumina-reinforced aluminum. Fiber-reinforced composites contain fiber materials (glass, carbon) in a polymer matrix (polyester, vinylester). The function of the fibers is to provide tensile strength and stiffness, whereas the polymer matrix transfers the applied force to the fiber and also provides high compressive strength.

Nanomaterials

Materials that are in nanoscale (less than 100 nm; $1\ \text{nm} = 10^{-9}\ \text{m}$) are called nanomaterials. This classification is based on the size but not the chemical makeup. Hence, nanomaterials can be metals, ceramics, polymers, or composites. As the particle/grain size becomes smaller, the properties of the materials may change drastically. For example, electrical insulators become conductors, chemically stable materials become combustible, and opaque materials become transparent. Although nanomaterials have been widely researched for medical use, they have also found applications in the construction industry, such as durable concrete and self-cleaning windows. The mechanical properties of concrete can be improved by adding a small amount of nano-silica or carbon nanotubes. Nanopowder of titanium dioxide is added to paints for its self-cleaning (break down dirt), sterilizing (inhibit bacterial growth), and excellent reflective (block UV light) properties. Nanocomposite coatings also provide corrosion, heat, and fire protection.

2.5 Imperfection in Materials

As discussed earlier in this chapter, crystals are repeated patterns of unit cells. However, there is nothing called a perfect crystal; imperfections in a crystal are unavoidable. In fact, imperfections are deliberately introduced in crystals to improve certain properties of materials. Imperfections in materials can be classified into three groups: (1) point defects, (2) line defects, and (3) surface defects.

Point Defects

Vacancies, interstices, and impurities are examples for point defects in a crystal (Figure 2.13). The internal energy of the crystal will increase, which in turn affects the properties of the material, when there are point defects.

Vacancies exist when there are vacant atomic positions within a crystal lattice. They play an important role in diffusion (movement) of atoms in the crystal lattice. They are generally created during solidification of molten material. The equilibrium number of vacancies N_v for a given quantity of material can be calculated using this relationship:

$$N_v = n\, e^{\frac{-Q_v}{kT}} \tag{2.3}$$

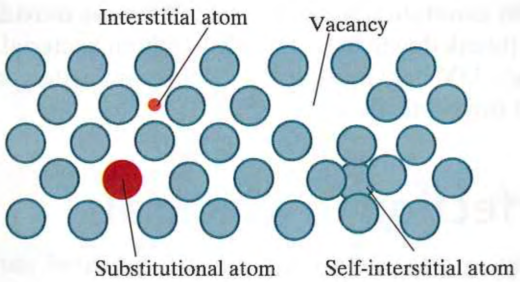
n—total number of atomic sites
Q_v—energy required for the formation of a vacancy (eV/atom)
T—absolute temperature in kelvin
k—Boltzmann's constant 8.62×10^{-5} eV/atom

Atoms, generally small atoms, can take the place of the interstitial site in a crystal lattice. For example, in steel (iron-carbon alloy), carbon atoms with a radius of 0.071 nm occupy the interstitial sites of iron atoms (radius 0.124 nm). When the parent atom takes the interstitial site, it is called a self-interstitial atom; however, the probability is less due to the large size of the parent atom. In many alloys, a foreign atom occupies the position of the parent atom, and this is called a substitutional atom. Substitutional atoms are usually close in size to the parent atom, within approximately 15% difference in atomic size. For example, in brass (copper-zinc alloy), zinc atoms, with a radius of 0.133 nm, take some of the positions of copper atoms, which have a radius of 0.128 nm.

Line Defects

Dislocation is a line defect and is caused by localized lattice distortion. Dislocations play a major role in the plastic deformation (irreversible change in shape when force or stress is applied) of materials, especially in metallic materials. Elastic strain fields

Figure 2.13 Different point defects in materials

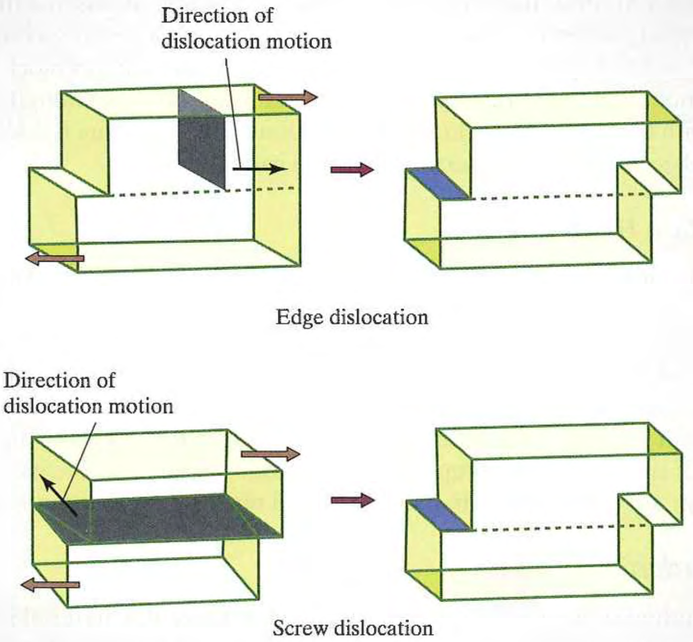


and stresses are associated with dislocations and can form in a crystal due to thermal stresses and phase transformations. Dislocations also affect the electrical property of materials. For example, the electrical resistance of pure copper decreases with an increase in dislocation density. There are three types of dislocations: edge, screw, and mixed dislocations. In an edge dislocation, localized lattice distortion exists along the end of an extra half-plane of atoms, meaning that the dislocation line moves parallel to applied stress. A screw dislocation results from shear distortion where the dislocation line moves perpendicular to applied stress (Figure 2.14). In crystalline materials, many dislocations have both edge and screws components, which are called mixed dislocations.

Surface Defects

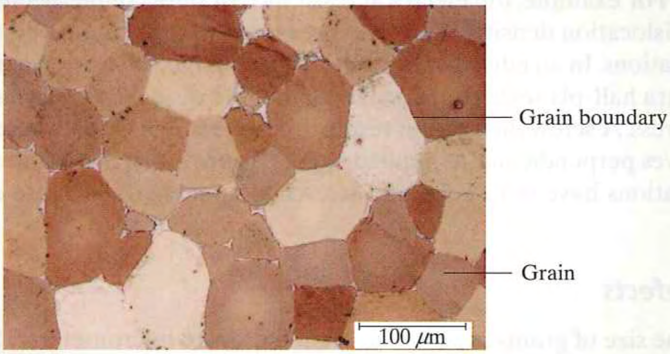
Generally, the size of grains ranges from nanometers to micrometers. The boundary between the grains is called a grain boundary, which is a surface defect (Figure 2.15). The atoms at the grain boundaries are imperfect (not aligned). Based on the orientation between two adjacent crystals, the grain boundaries are named as low-angle grain boundary and high-angle grain boundary (>10−15°). Grain boundaries, which limit the length and motion of dislocations, influence the mechanical properties of materials. The external surface or free surface of a crystal is also known as a surface defect.

Direction of dislocation motion in edge and screw dislocations. **Figure 2.14**



(Adapted from Callister W. D., and Rethwisch D.G. 2010)

Figure 2.15 Optical microstructure of an alloy showing grains and grain boundaries.



(Courtesy of M. Bobby Kannan, James Cook University, Australia)

2.6 Strengthening in Materials

The traditional ways of strengthening materials are: (1) solid-solution strengthening, (2) precipitation hardening, (3) strain hardening, and (4) grain refinement.

Solid Solution Strengthening

An alloy is basically made of solute and solvent atoms. The size of the solute atoms can be smaller or larger than the solvent atoms. Due to the difference in size, a stress field is created around the solute atoms. When the solute atoms are smaller (as in interstitial solid solution), a tensile stress field is created, whereas a compress stress field is produced when the solute atoms are larger. The stress fields block the dislocation movement and consequently strengthen the material. The strengthening effect increases with the increase in solute concentration. However, there is a solubility limit above which segregation (clustering of atoms) can take place.

Precipitation Hardening

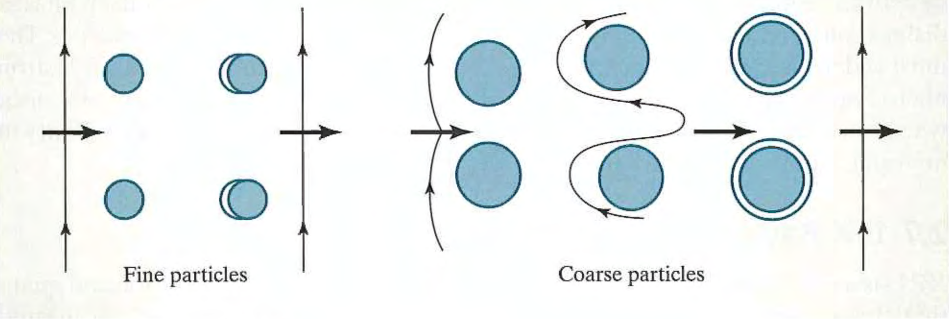
The strength of a metal can be improved by inducing precipitation. The size of the precipitates dictates the strengthening mechanism (Figure 2.16). For example, fine precipitates (<50 Å (angstrom); $1\ \text{Å} = 10^{-10}\,\text{m}$) act as obstacles for moving dislocations, and the dislocations cut through the fine precipitates. In the case of coarse precipitates (100–500 Å), the dislocations bend and bypass them. Comparatively, fine precipitates are more effective than coarse precipitates in improving the strength of a material. If the spacing between the precipitates is larger, as in the case of a coarse precipitation, the dislocation can pass easily and results in less strengthening.

Strain Hardening

Another method of increasing the strength and hardness of a material is by increasing the dislocation density. This can be achieved by cold working the material, and this process of hardening is called strain hardening. Typically, annealed crystals have

Dislocation motions in a material containing fine and coarse precipitates.

Figure 2.16



(Adapted from Raghavan V., 1997)

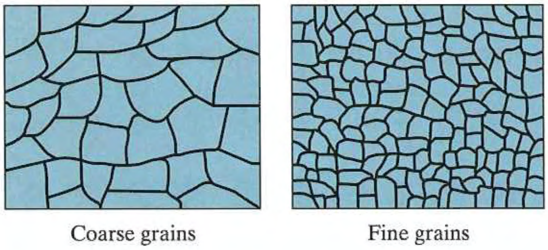
a dislocation density of $\sim 10^8 \text{m}^{-2}$, which can be increased to $10^{10}-10^{12} \text{m}^{-2}$ by moderate cold working and to $10^{14}-10^{16} \text{m}^{-2}$ by heavy cold working. Although dislocations improve ductility of a material, high dislocation density will obstruct dislocation movements, and they interfere with each other and make the material brittle.

Grain Refinement

Grain boundaries in a material act as obstacles for dislocation movement. Hence, an increase in the grain boundary area will consequently increase the strength of the material. One of the reasons for the popularity of nanomaterials is their high strength, which is due to the high grain boundary area in nanomaterials. For a given volume, finer grains exhibit larger grain boundary area as compared to that of coarser grains (Figure 2.17). The grain size of materials can be controlled by altering the cooling rate during the casting process or by heat treatments. In general, fast cooling produces smaller grains, and larger grains are formed under slow cooling. Further, fine grains can be produced by alloying and/or thermomechanical treatments.

Schematic showing large grain boundary area (black lines) in the microstructure containing fine grains as compared to the microstructure containing coarse grains

Figure 2.17


Coarse grains
Fine grains

2.7 Characterization of Materials

Microscopy has played a major role in the development of advanced materials. Microstructural features such as grain size and shape, distribution of secondary phases, dislocations, and inclusions can be identified and quantified, using microscopes. The most widely used microscopes for materials characterization are optical and electron microscopes. The atomic force microscope (AFM) is a recent addition to microscopes, which can be used to study the topography of materials at nanoscale. Crystallinity in materials can be identified using x-ray diffraction (XRD) technique.

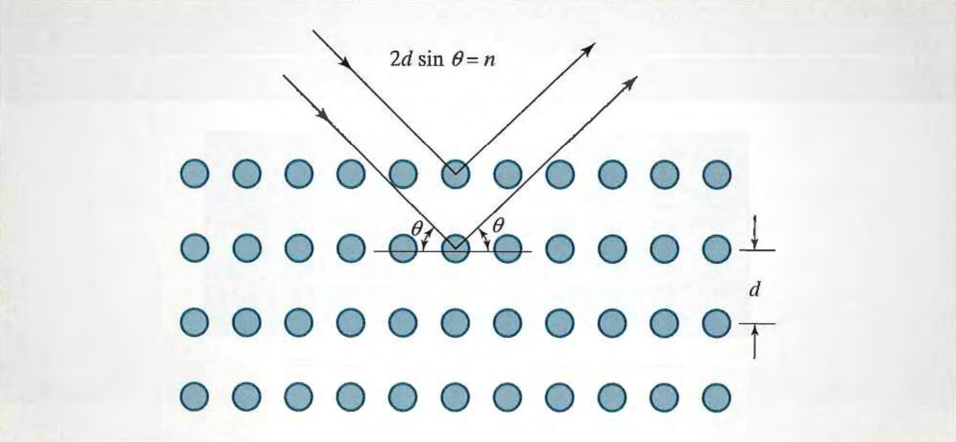
2.7.1 X-Ray Diffraction (XRD)

XRD is a nondestructive analytical technique used for phase identification and quantification of a crystalline material. In principle, the sample in the form of fine ground powder or a flat surface is illuminated with X-rays of a fixed wavelength and intensity, and a goniometer is used to record the reflected radiation. The distances between the planes of the atoms of the samples can be calculated by applying *Bragg's Law*, which states that $2d \sin \theta = n\lambda$, where the integer n is the order of diffracted beam, the wavelength of the incident x-ray beam is λ, d (also known as d-spacing) is the distance between adjacent planes of atoms, and θ is the angle of incidence of the X-ray beam (Figure 2.18). Using this technique, a wide range of materials can be tested, including metals; fine-grained minerals such as clays, composites, inorganic and organic compounds; and polymers. The purity of the materials can also be determined. A photograph of an XRD equipment and a typical XRD spectra for a crystalline material are shown in Figure 2.19.

2.7.2 Optical Microscope

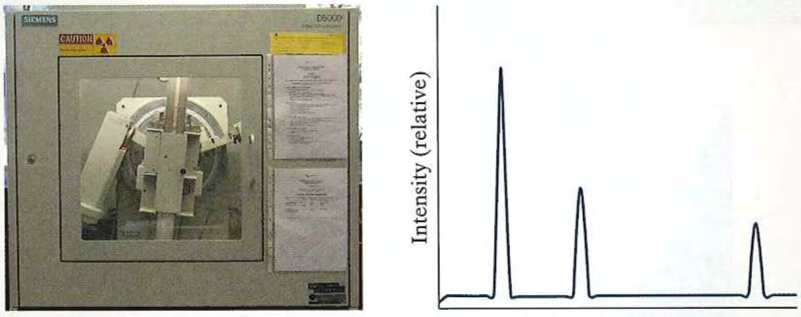
The simplest of all microscopes is the optical microscope (Figure 2.20). The source of illumination is a visible light, and its operation is based on the reflection of light. The light passed through the objective lenses is reflected back by the metal surface and partly transmitted through the reflector to the eyepiece. The surface of the material

Figure 2.18 Scattering of X-rays by crystallographic planes



(a) Photograph of an X-ray diffraction (XRD) equipment; (b) XRD spectra of a crystalline material

Figure 2.19

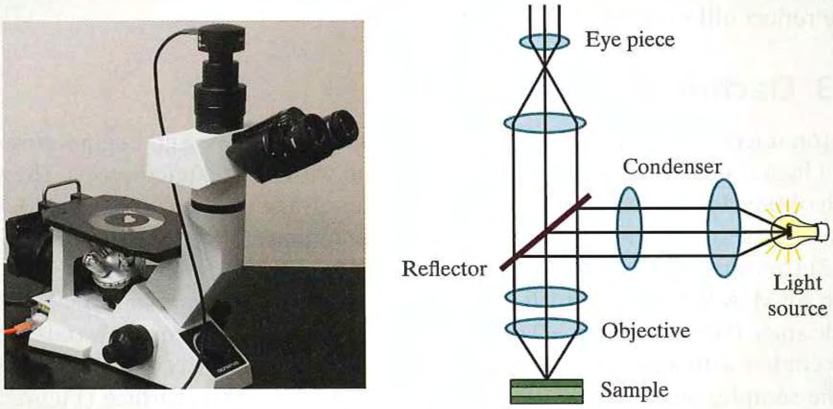


(a) (b)

(Courtesy of M. Bobby Kannan, James Cook University, Australia)

Photograph and principle of optical microscope

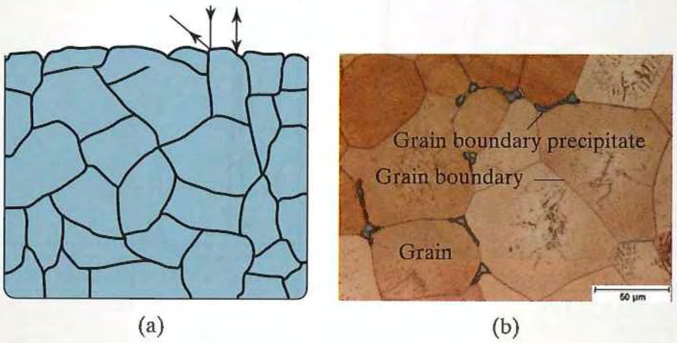
Figure 2.20



(Courtesy of M. Bobby Kannan, James Cook University, Australia)

should have a smooth surface finish to examine the microstructure of the material. Generally, the smooth surface is achieved by grinding the material, using polishing papers (silicon carbide), followed by fine polishing using alumina or diamond paste. A chemical reagent known as *etchant* is used to mildly corrode the materials to reveal the grain boundaries. Due to the high energy of the grain boundaries, they undergo corrosion quickly as compared to the interior grains such that the grain boundaries become rough and the grains remain smooth. Sometimes, the precipitates are also partially or completely dissolved due to the etchant. Hence, the grains reflect the light, but not the grain boundaries or the precipitates; as a result, the grains appear bright and the grain boundaries and precipitates are dark under an optical microscope (Figure 2.21). In some materials, the grains are also undergo corrosion to some

Figure 2.21 (a) Chemical etching produces grooves along the grain boundaries, and the optical light that falls on the grooves will not reflect back into the objective lens; (b) An optical micrograph of an alloy exhibiting grains, grain boundaries and precipitates



(Courtesy of M. Bobby Kannan, James Cook University, Australia)

degree, depending on their orientation with respect to the plane of observation, and hence reflect different amounts of light.

2.7.3 Electron Microscopes

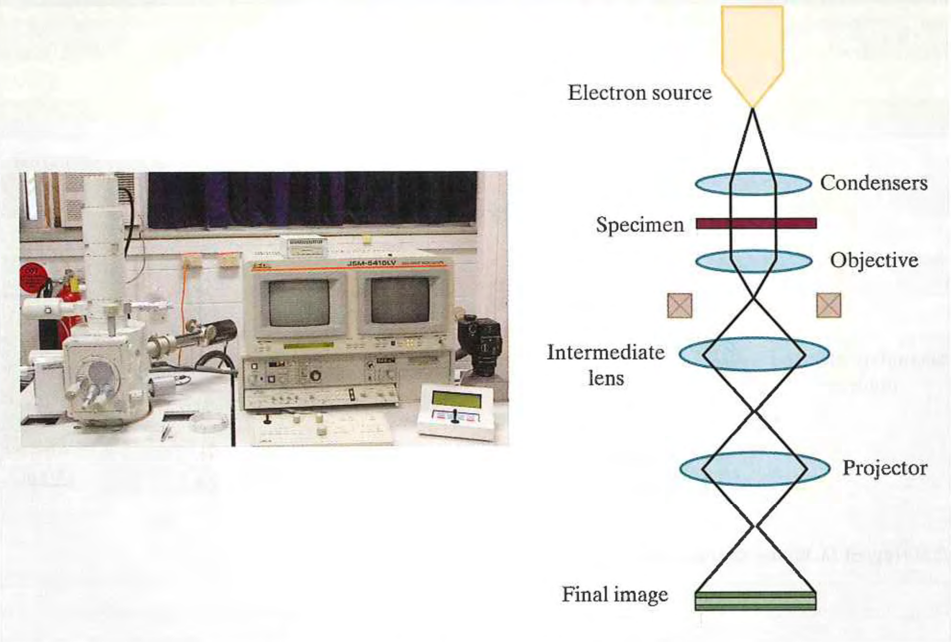
Electron microscopes are powerful tools for material science and engineering. They have a higher power of resolution in comparison with optical microscope. The wavelength of the electron beam is about 0.05 Å, whereas for visible light it is 5000 Å. The two main types of electron microscopes are (1) scanning electron microscopes (SEM), and (2) transmission electron microscopes (TEM).

In SEM, solid samples such as metals, ceramics, or polymers are placed in the sample stage (vacuum chamber) of the equipment. Nonconducting materials are generally coated with a thin film of carbon or gold for conductivity. The electron beam hits the sample, and a variety of signals are emitted from the surface (Figure 2.22). The three main signals that provide useful information are the secondary electrons, backscattered electrons, and X-rays. Secondary electrons emitted from the top surface of the sample produce the topography image of the sample. Due to the high depth of focus in SEM, fracture surface analysis can be done using this equipment (Figure 2.23). Backscattered electrons are the primary beam electrons, which are reflected from the surface, and produce contrast in the image when the sample contains elements with appreciable difference in atomic numbers. Backscatter image can hence provide the distribution of different chemical phases in the sample. Using the emitted X-rays, elemental analysis (energy dispersive X-ray spectroscopy, EDS) such as mapping or line profile of elemental distribution can be done.

In TEM, the electron beam is transmitted through the samples (Figure 2.24). Basically, TEM is composed of an electron gun for generation of electron beam and a vacuum system for the electrons to travel through a series of electromagnetic lenses and the sample. The objective lens focuses the electron beam that transmits through the samples onto an image on a phosphor screen. Dark regions of the image are due

Photograph and principle of scanning electron microscope (SEM)

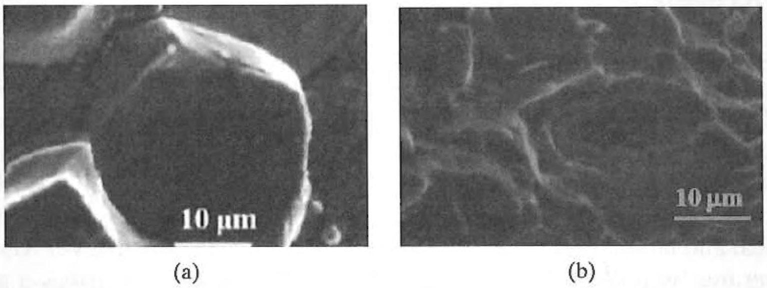
Figure 2.22



(Courtesy of M. Bobby Kannan, James Cook University, Australia)

Scanning electron micrographs of fractured metal samples (also called fractography): (a) brittle failure (intergranular cracking) and (b) ductile failure

Figure 2.23

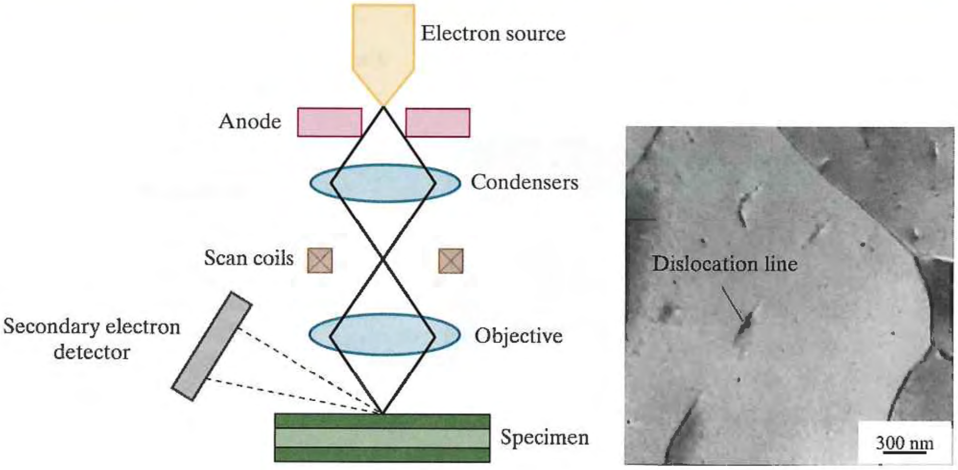


(a) (b)

(Courtesy of M. Bobby Kannan, James Cook University, Australia)

to transmission of only a few electrons, whereas the light regions are a result of more electrons transmitted through the sample. The sample preparation for TEM analysis is tedious since a very thin sample (less than 2500 Å thick) is required for the transmission of electrons through the sample. There are a few methods, such as chemical etching and ion milling, to prepare the sample for TEM analysis. *Chemical etching* is carried out, particularly for metallic samples, to obtain the required thickness. In the

Figure 2.24 (a) Principle of transmission electron microscope (TEM); (b) a TEM micrograph of an alloy revealing dislocation lines



(a) (b)

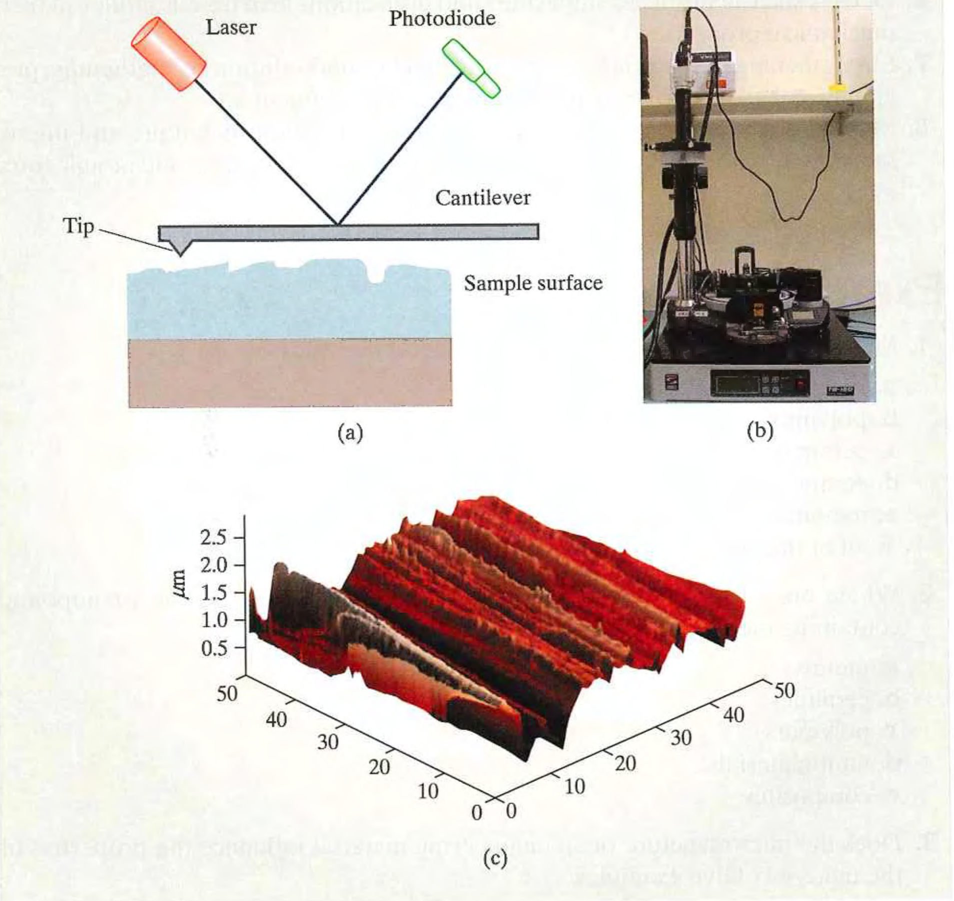
(Courtesy of M. Bobby Kannan, James Cook University, Australia)

ion-milling process, charged argon ions are accelerated to the surface of the sample by use of high voltage, where the ion impingement removes material from the sample surface. Using TEM, detailed microstructural features such as dislocations, crystal structure, grain boundaries, and chemical analysis on a nanoscale can be analyzed.

2.7.4 Atomic Force Microscope (AFM)

AFM is a form of scanning probe microscope that has a very high resolution and provides a three-dimensional surface profile on a nanoscale (Figure 2.25). Basically, an AFM consists of a cantilever with a sharp tip, which is a piezoelectric material, usually silicon or silicon nitride with approximately 100Å diameter, at its end that is used to scan the sample surface. During the scanning, the forces between the tip of the cantilever and sample lead to a deflection of the cantilever. The AFM measures the vertical and lateral deflection of the cantilever using an optical lever. The optical lever operates by reflecting a laser beam off the cantilever that strikes a position-sensitive photodiode. AFM can be used to study the topography of a wide variety of materials, including metals, ceramics, and polymers. One of the main advantages of AFM is that it does not required vacuum; it can be operated in ambient air or even in a liquid environment. Also, there is no need for any special sample preparation such as conductive coatings or chemical treatments. Both conducting and nonconducting materials can be analyzed using AFM. Using a special probe, the hardness of different phases (small size) in materials can be measured, which is generally not possible with conventional hardness testing equipment.

Figure 2.25 (a) Principle of atomic force microscope (AFM); (b) Photograph of an AFM; (c) A topography image of a rough surface alloy obtained using an AFM



(Courtesy of M. Bobby Kannan, James Cook University, Australia)

2.8 Summary

1. The nucleus of an atom contains protons and neutrons. Electrons are contained in shells around the nucleus.
2. There are three types of primary bonds: metallic, covalent, and ionic, which are relativity strong. A van der Waals bond is a weak secondary bond which exhibits in some materials.
3. Materials which have long-range atomic order are called crystalline materials, and the materials with disordered atomic distribution are called noncrystalline or amorphous materials.
4. There are basically seven types of crystal systems. The three most common crystal structures are body-centered cubic (BCC), face-centered cubic (FCC), and hexagonal close-packed (HCP).

5. Materials can be broadly classified based on chemical nature into metals, ceramics, and polymers. Composites and nanomaterials are advanced materials with attractive properties.
6. Defects such as vacancies, impurities, and dislocations in materials influence their mechanical properties.
7. Strengthening of materials can be achieved by solid solution strengthening, precipitation hardening, strain hardening, and grain refinement.
8. Materials can be characterized using X-ray diffraction technique and microscopes, such as optical, scanning electron, transmission electron, and atomic force microscopes.

Exercises

1. Materials science and engineering include(s) the study of
   a. metals
   b. polymers
   c. ceramics
   d. composites
   e. nanomaterials
   f. all of the above
2. Which one of the following engineering materials is defined as a compound containing metallic and nonmetallic elements:
   a. metals
   b. ceramics
   c. polymers
   d. nanomaterials
   e. composites
3. Does the microstructure of an engineering material influence the properties of the material? Give examples.
4. Why are metals good conductors of electricity?
5. Nanomaterials is a class of materials based on:
   a. chemical structure
   b. mechanical properties
   c. density
   d. color
   e. size
6. How are atoms arranged in crystalline and amorphous materials? Comment on the density of these materials.
7. What is an alloy? Give examples.
8. Explain why ceramics have higher melting points than polymers.
9. Define valence electron. What is the significance of valence electron(s)?
10. All crystalline solids contain vacancies. (True/False)
11. Self-interstitials are highly probable in solids. (True/False)

12. What is the difference between solid solution and second phase particles in an alloy?
13. Calculate the theoretical density of aluminium (FCC) and chromium (BCC).
14. Gold has an atomic radius of 0.1442 nm and a density of 19.3 g/cm^3. Determine whether it has a BCC or FCC crystal structure.
15. Suggest a suitable microscope for fracture-surface analysis.
16. Explain how imperfections in materials alter their properties.
17. Comment on the influence of dislocations in metals on their ductility.
18. Suggest two methods to decrease the grain size of a material for improved mechanical properties.
19. List the advantages of AFM as compared to TEM.
20. Suggest a method to identify crystallinity in materials.

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3 CHAPTER

It is necessary to determine the soil properties through laboratory and field tests shown here. The drill makes boreholes at the site for collecting soil samples to determine the soil properties. Courtesy of N. Sivakugan, James Cook University, Australia

Soils

3.1 Introduction

Soils, rocks, and aggregates are natural materials of geologic origin, fondly known as *geomaterials*. They are the most abundant materials on earth and are also of low cost. They have been used as construction materials for centuries, starting from early civilization. These three geomaterials are discussed separately in chapters 3, 4, and 5.

While sand-sized and gravel-sized aggregates are commonly used in concrete and roadwork, very fine-grained soils such as silts and clays have limited use as construction materials. Our interest in soils is not as a construction material but more as a material on which buildings, bridges, and dams are built. It is in soils we carry out excavations to bury pipelines or construct basements. Earth- and rock-filled dams are made entirely of geomaterials. The objective of this chapter is to give a broad overview of the engineering behavior of soils, leaving the nitty-gritty and analysis to the *soil mechanics* or *geotechnical engineering* courses in your degree program.

When designing a beam or a bridge, the designer has the luxury of specifying the strength of concrete. The same thing applies for most engineering materials. When it comes to soils, the situation is different. Once the site is identified, one has to design the structure to suit the soil conditions. While some form of *ground improvement* is possible, any attempt to replace the soil with a better performing soil can be an expensive option.

3.2 Civil Engineering Applications

All structures and facilities (for example, buildings, bridges, highways, and tunnels) have to be placed on the ground and need some form of foundation. They also require site preparation and earthwork during the early stages of the project. Some of the traditional geotechnical engineering applications that require working with soils include foundations, retaining walls, dams, and excavations.

Soils are also used as construction materials in roadwork and embankments and as backfills behind retaining walls and in trenches where pipelines are buried. With a limited supply of virgin geomaterials freely available, they often have to be quarried or sourced from distant places, thus increasing the cost. With the increasing focus on sustainability in engineering, there have been attempts to use construction and demolition waste, recycled concrete, fly ash, slag, and other waste products as a substitute for soils and aggregates.

In the recent years, there have been attempts to use small amounts of lime or *supplementary cementitious materials* (SCM) such as fly ash, slag, and silica fume in soils to improve their properties. These SCMs are *pozzolanic*, rich in silica, and cost significantly less than cement. Pozzolanic materials are those that can react chemically in the presence of water, forming compounds with cementitious properties. The SCMs can be used as partial replacement of the ordinary portland cement. Fly ash is a form of dust collected from the precipitators in coal-burning power stations. Slag is the waste product in blast furnaces, used for smelting iron ore. Silica fume is a byproduct of silica smelting. It is an ultrafine material with spherical grains less than 1 μm in diameter. Using the SCMs as a binder is also a good way of disposing what is otherwise seen as a waste product. Lime is derived from limestone rocks.

3.2.1 Traditional Geotechnical Applications

Figure 3.1 shows some traditional geotechnical engineering applications for working primarily with soils. In most of these applications, the two major concerns are *failure* and *deformations*. While it is necessary to ensure that the soil around the structure does not fail, it is also necessary that the deformations remain within tolerable limits. Failure relates to the strength of the geomaterials, and the deformations relate to deformability parameters such as soil modulus.

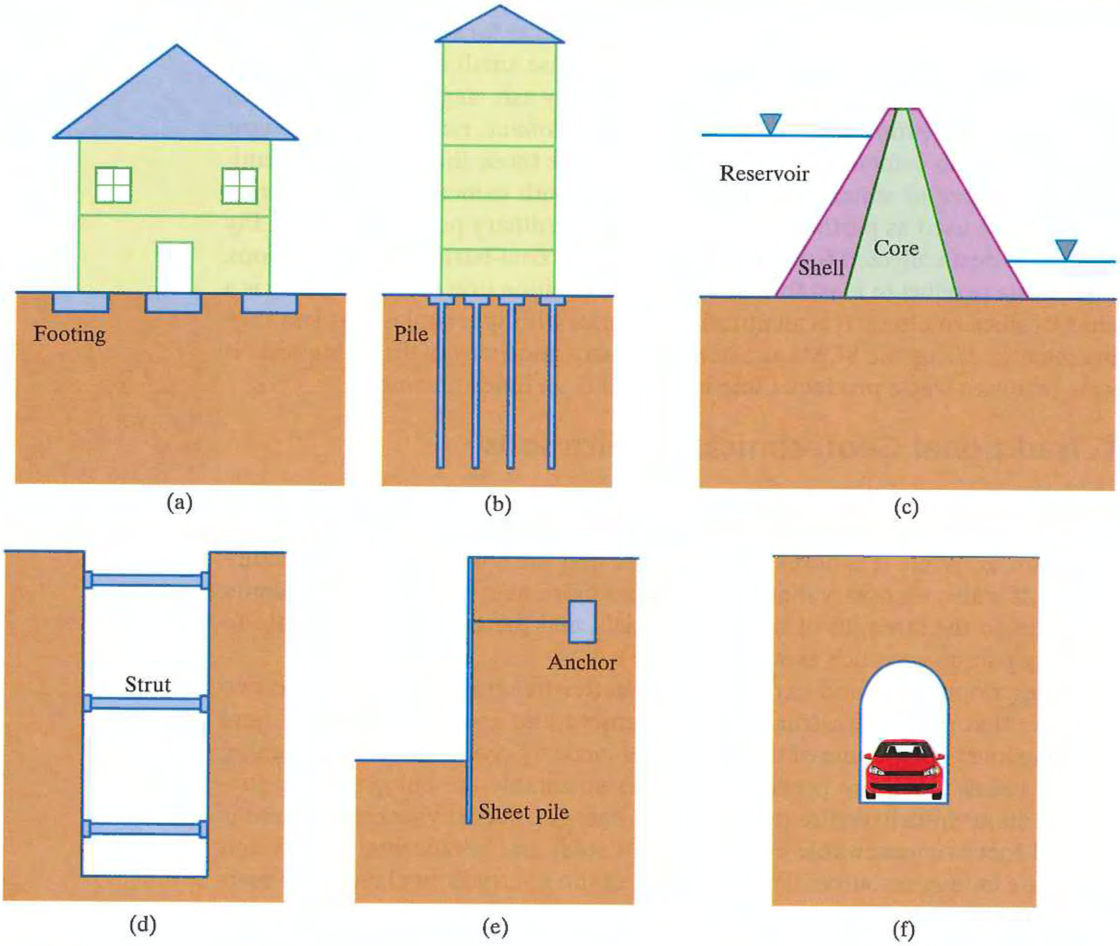
Increasing population and expectation of better living standards are the two major factors that drive infrastructure development and growth. Scarcity of land and natural resources are some of the challenges faced by civil engineers in meeting these needs. Lands that were previously seen as unsuitable are being utilized successfully for major infrastructure projects. With energy prices skyrocketing, there is an increased focus on renewable energy, such as solar and geothermal energy and other alternate energy resources. Through some of the nontraditional civil engineering applications, we also create new problems and get into uncharted territories. During the past few decades, *environmental geotechnology* has become an emerging area that deals with interaction between the earth and the environment.

3.2.2 Backfilling Underground Mines

Mining is a multibillion dollar industry in many countries including the United States, Canada, South Africa, Brazil, Australia, China, and India. *Open pit* and *underground mining* are two different ways of mining for shallow and deep ore bodies, respectively. When ore is removed from the ground for extracting minerals, very large voids are created in the ground. These are usually backfilled with the crushed waste rock, which is available in large quantities after the minerals are removed from the ore.

Mine backfilling is an effective way of disposing the crushed waste rocks or tailings, and it improves the stability of the region for subsequent mining in the nearby areas. Some of the common types of mine backfills are hydraulic fill, paste fill, sand fill, aggregate fill, and rock fill, all derived from crushed waste rock, differing only in the grain-size distribution. These backfills can be mixed with a binder, such as ordinary portland cement (OPC) or other SCMs, to increase their strength.

Figure 3.1 Some traditional geotechnical engineering applications: (a) shallow foundations; (b) deep foundations; (c) earth dam; (d) braced excavation; (e) anchored sheet pile; and (f) tunnel



3.2.3 Land Reclamation Using Dredge Spoils

Ports play a key role in economic development worldwide. To ensure navigable depths for the oncoming and outgoing vessels, ports are obliged to carry out regular *maintenance dredging* and some *capital dredging* (for example, creating a new waterway). In the United States, more than 230 million tonnes of sediments are removed from sea/river beds annually, with similar quantities reported from Europe.

Dumping the dredge spoil into the sea can lead to serious environmental concerns, such as damage to aquatic habitats, reduction in water quality, and disruption to ecosystems. As a result, many countries have strict measures in place to limit offshore dumping of dredge spoils. As a step in the right direction toward sustainability, these days dredge spoil is seen more as a resource than waste. It is being used in land reclamation, beach nourishment, and as an engineered fill material.

Land reclamation: (a) containment paddocks; (b) dredge spoil pumped into the paddock

Figure 3.2

(a) (b)

(Courtesy of N. Sivakugan, James Cook University, Australia)

Figure 3.2 shows land reclamation carried out at the Port of Brisbane, Australia, where the enclosure within a sea wall is subdivided into containment paddocks which are filled with the dredge spoil in the form of slurry. The slurry settles and consolidates into reclaimed land for future use.

3.3 Formation of Soils

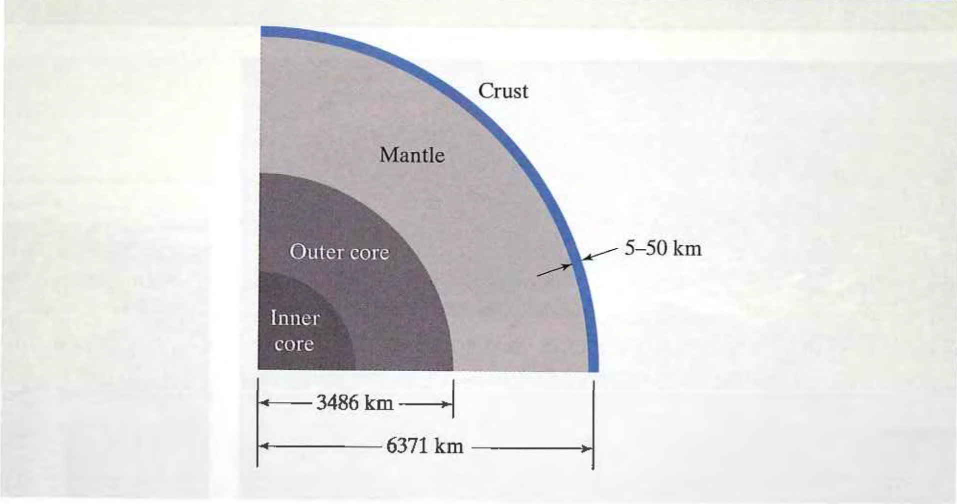
Soils were formed over thousands of years when the parent rocks broke down into smaller fragments which broke down further, in a process known as *weathering*. Although weathering is broadly categorized as *physical weathering* or *chemical weathering*, the breakdown is triggered by several physical (for example, burrowing animals, plant roots), chemical (acidic leachates, oxidation, hydration), biological (bacteria, algae), and geological agents (water, ice, air).

Soils are made primarily of silicates (oxides of silicon) and oxides of other elements such as Al, Fe, Ca, Mg, Na, and K. The silicates include feldspar, quartz, and others. Other than silicates, there are carbonates, such as limestone ($CaCO_3$); sulphides, such as galena (PbS); sulphates, such as gypsum ($CaSO_4{\cdot}2H_2O$); halides, such as salt (NaCl), and native elements, such as gold (Au), copper (Cu), silver (Ag), and others.

3.3.1 Elements of Earth

Earth has the shape of a spheroid (not exactly a sphere), with an *average* diameter of 12,742 km. The major elements of the entire earth are iron (32.1%), oxygen (30.1%), silicon (15.1%), and magnesium (13.9%). It is made of a solid crust at the surface, followed by a very viscous mantle, and a liquid outer core and a solid inner core (Figure 3.3). While geologists are interested in what lies beneath, right to the center of the core, civil engineers are interested mainly in the crust that extends 5–50 km beneath the surface. The deepest mines and underground tunnels on earth are within

Figure 3.3 Cross section of the earth



the top 5 km of the crust. The *oceanic crust* is typically 5–10 km in thickness, and the *continental crust* can be 30–50 km in thickness.

The composition of the crust is quite different from that of the entire earth. About 98% of the crust is made of eight elements, namely O (46.6%), Si (27.7%), Al (8.1%), Fe (5.0%), Ca (3.6%), Na (2.8%), K (2.6%), and Mg (2.0%). These are the eight main elements that are present in *rock-forming minerals*. A mineral is a naturally occurring, inorganic, solid, and crystalline substance with a certain structure and chemical composition. These minerals can be broadly categorized as *silicates* and *nonsilicates*. Silicates are made primarily of silicon and oxygen, and more than 90 percent of the earth's crust is made of silicates. Only 5–8 percent of the earth's crust is made of nonsilicate minerals.

Silicates can be grouped into one of the following seven groups, where feldspar, quartz, and mica are the three predominant rock-forming minerals in their order of abundance.

- Feldspar (examples: orthoclase [$KAlSi_3O_8$, plagioclase [$NaAlSi_3O_8$, $CaAl_2Si_2O_8$])
- Quartz (example: quartz [SiO_2])
- Mica (examples: biotite [$K(Mg, Fe)_3AlSi_3O_{10}(OH)_2$], muscovite [$KAl_2(AlSi_3O_{10})(OH)_2$])
- Pyroxene group (example: augite [$(Ca, Na)(Mg, Fe, Al, Ti)(Si, Al)_2O_6$])
- Amphibole (example: hornblende [$(Ca_2(Mg, Fe)_4Al(AlSi_7O_{22})(OH)_2$])
- Olivine group (example: olivine [$(Mg, Fe)_2SiO_4$])
- Kaolinite (example: kaolinite [$Al_2Si_2O_5(OH)_4$])

Nonsilicate minerals can be grouped under one of the following:

- Carbonates (examples: calcite [$CaCO_3$], dolomite [$CaMg(CO_3)_2$])
- Sulfides (examples: galena [PbS], pyrite [FeS_2]) and sulphates (examples: gypsum [$CaSO_4{\cdot}2H_2O$], anhydrite [$CaSO_4$])
- Oxides (examples: hematite [Fe_2O_3], magnetite [Fe_3O_4], corundum [Al_2O_3], rutile [TiO_2], uranite [UO_2])
- Native elements (examples: gold [Au], silver [Ag], platinum [Pt], copper [Cu])
- Evaporites (examples: halite [NaCl], sylvite [KCl])

3.3.2 Igneous, Sedimentary, and Metamorphic Rocks

Rocks can be grouped under three broad categories *igneous rocks, sedimentary rocks*, and *metamorphic rocks*, depending on the geological process associated with their formation. Earth's crust consists of approximately 65 percent igneous rocks, 27 percent metamorphic rocks, and 8 percent sedimentary rocks.

Igneous Rocks

It gets very hot inside the earth, where the temperature can exceed 1000 °C. Molten rock beneath the surface is called *magma*, and when it comes above the surface, it is known as *lava*. They are literally the same in composition. Magma/lava consists of elements such as Si, Fe, Na, K, and so forth. When it cools below the surface, the cooling is a slow process, taking thousands of years, and the grains are coarse, can be seen by the naked eye, and hence are easier to identify. Such rocks are known as *intrusive* or *plutonic*. Some examples of intrusive rocks are granite, diorite, gabbro, and peridotite. When the lava cools on the surface, the process is fast, taking a few days to weeks, not giving much time for the grains to grow. Here, the grains are fine and are difficult to identify with the naked eye. These rocks are known as *extrusive* or *volcanic*. Some examples of extrusive rocks are rhyolite, andesite, basalt, and komatite. Basalt and granite are the two most common igneous rocks. Figure 3.4 shows a granite rock that was intruded by black basalt dyke.

Most igneous rocks are made of eight rock-forming minerals: quartz, orthoclase feldspar, plagioclase feldspar, muscovite mica, biotite mica, amphibole, pyroxene, and olivine. The first four are light colored and are known as *felsic* (refers to feldspar and silica rich) minerals. They typically contain more than 62 percent SiO_2 and are acidic. The last four are dark colored and are known as *mafic* (refers to magnesium and iron rich) minerals, which are basic. Depending on the mineral composition, the igneous rocks, whether volcanic or plutonic, can be classified as felsic, intermediate,

Granite intruded by black basalt dyke

Figure 3.4

(Courtesy of N. Sivakugan, James Cook University, Australia)

Table 3.1 Composition of igneous rocks

Group name % of Silica	(Felsic) >65	(Intermediate) 55–65	(Mafic) 45–55	(Ultramafic) <45
Color	Light (white, pink)		Dark	
Intrusive rock	Granite	Diorite	Gabbro	Peridotite
Extrusive rock	Rhyolite	Andesite	Basalt	Komatite (rare)

mafic, and ultramafic. Granite and rhyolite are felsic, gabbro and basalt are mafic, and peridotite is ultramafic. Generally, igneous rocks are poorer in silica and richer in ferromagnesium silicates.

Depending on the composition of magma and the rate of cooling, different types of igneous rocks are formed. The relative proportion of silica dictates how the igneous rocks are described, based on the composition, as light-colored *felsic* rocks or darker *mafic* rocks (see Table 3.1).

During the geologic times, the igneous rocks (and other rocks, too) broke down into fine *sediments* through *mechanical weathering*. The sediments undergo ion exchanges through *chemical weathering*, where their composition can change. These sediments are transported to other locations by gravity, wind, water, air, or ice. When transported by water or wind, they are sorted nicely by size, and the grains become more rounded. When transported by gravity or ice, little or no sorting take place and the grains are generally angular. These sediments get deposited, forming layers that get cemented and become *sedimentary rocks*.

Sedimentary Rocks

While sedimentary rocks make up only about 8 percent of the earth's crust, their presence on the surface of the earth can be substantial. They are formed by deposition of geomaterials on the earth's surface and in water bodies. The sedimentary rocks can be broadly categorized as *detrital (clastic)* or *chemical sedimentary rocks*. Detrital sedimentary rocks are made of broken fragments of preexisting parent rocks and hence retain the properties of the parent rocks. For example, clay becomes *shale*, silt becomes *siltstone*, sand becomes *sandstone*, and gravels and boulders become *conglomerates* (rounded grains) or *breccia* (angular grains). The chemical sedimentary rocks are formed by a chemical process, where the minerals are dissolved in water and form new minerals. These include *limestone, dolomite, chert* [SiO_2], *rock salt* (NaCl), *gypsum*, and *coal*. Shale is the most abundant type of sedimentary rock.

Metamorphic Rocks

Metamorphic rocks are formed from preexisting igneous, sedimentary, or metamorphic rocks, which undergo major structural change caused by high levels of pressure and heat. Low-grade metamorphic rocks (for example, *slate*), which form under relatively low pressures and/or temperatures, retain the characteristics of the parent rocks. High-grade metamorphic rocks (for example, *gneiss*), which are subjected to high temperatures and pressures, can be very different from their parent rocks. At low temperatures and pressures, shale transforms into slate; at intermediate pressures and temperatures, shales transform into *phyllites*, and at very high temperatures and pressures, it transforms into schist or gneiss. Limestone ($CaCO_3$) and dolostones (dolomite rocks [$CaMg(CO_3)_2$]) transform into *marble*. Due to the high temperatures and pressures, some of the minerals get elongated and form sheets, occurring in thin layers. This is called *foliation*. Slate, phyllite, schist, and gneiss are examples of foliated metamorphic rocks. *Marble* and *quartzite* are nonfoliated metamorphic rocks. In general, igneous rocks are more isotropic than sedimentary or metamorphic rocks.

3.3.3 Residual and Transported Soils

Residual soils are formed by in situ weathering of parent rocks. These soils remain at the same location and retain the same characteristics as the parent rock. When the weathered sediments are carried away to distant locations by wind, gravity, water, or ice, they are known as *transported soils*. Depending on the geologic agent involved in the transportation process, they have special names.

Alluvial soils are transported by running water, such as rivers and streams (for example, sedimentary clays). Soils that are transported by wind are known as *aeolian soils* (for example, loess). *Colluvial soils* are those transported by gravity, such as in landslides and soils transported down slopes. *Glacial soils* are soils transported by glaciers (such as glacial till). Glaciers are moving ice masses that carry soil and rock fragments with them. *Lacustrine soils* are soils transported and deposited into freshwater lakes (for example, lacustrine clays). *Marine soils* are deposited in sea water (for example, marine clays). The transported soils are generally loose and soft to several meters of depth.

There are some soil types that we encounter in civil engineering, which have been assigned special names that are worth a mention here. These are summarized in Table 3.2.

Special names for soils

Table 3.2

Special name	Description
Boulder clay	An unstratified glacial deposit consisting of boulders and pebbles in a clay matrix.
Calcareous soil	A soil rich in calcium (or some magnesium) carbonate. Alkaline, with high pH.
Caliche	Sedimentary rocks consisting of cemented calcium carbonate, or a dense layer of soil where the grains are cemented together by calcium carbonate.
Hardpan	A very dense soil at shallow depths, which is difficult to excavate.
Laterite	A tropical soil rich in iron and aluminum, often red in color, formed by weathering of underlying parent rock.
Loam	A sand-silt-clay mixture that is ideal for agricultural purposes. Good drainage and nutrients.
Loess	Wind-blown silt of uniform grain size in the range 0.01–0.05 mm, yellowish brown in color. More permeable vertically than horizontally.
Marl	Stiff or very stiff marine calcareous clays and sands.
Peat	Partly decayed vegetation. Very compressible.
Shale	Soft rock or hard soil? In a state of transition from clay to slate. Formed in layers.
Till	Unstratified and poorly sorted glacial deposit of clay, silt, sand, gravel, and boulders. Often heavily overconsolidated. Boulder clay is a till.
Tuff	Rock made of consolidated volcanic ash produced in volcanic eruption.
Varved clay	Alternating layers of clays and silts deposited in glacial lakes (lacustrine soil). The varves are formed by seasonal variations.

3.4 Soils versus Other Engineering Materials

Soils are different from other engineering materials. Most engineering materials are *homogeneous*, showing the same properties at every location. Soils cover large areas and volumes and are often *heterogeneous*, meaning that they are nonhomogeneous. The soil composition and properties can vary within few meters, laterally or vertically.

Most of the engineering materials are *isotropic*, showing the same properties in all directions. This is not the case with soils, which are often *anisotropic*, where the properties vary with direction. For example, the permeability of soil is often greater in the horizontal direction than in the vertical direction.

Engineering materials are generally treated as a *continuum* (a material not separated into distinct grains) where most of the engineering mechanics theories (for example, Mohr circles, theory of elasticity discussed in chapter 1) can be applied. Soil is, however, a *particulate medium*, which is made of particles or grains that are not necessarily held together. The load transfer mechanism in a particulate medium is very complex. The soil matrix contains voids, which are filled with air and/or water. That makes the soil a three-phase (air, water, and solid) medium, making it more complex. Most other engineering materials are treated as single-phase materials. For simplicity, soils are generally assumed as continuous media in designs and analysis.

Engineering materials, such as concrete or steel, when subjected to external loads, can fail under tension, compression, or shear. In the case of soils, almost always the failure takes place in shear, where the grains slide over each other along the failure surface. The failure of individual soil grains in tension or compression is rarely a concern.

The stress-strain diagrams of soils are similar to those of strain-hardening or strain-softening materials shown in Figure 1.5. The strain hardening behavior is common in loose soil deposits, and the strain softening behavior is seen in dense ones. Under working loads, which are only a fraction of the failure loads, it is common to assume that soils behave as linear elastic materials. In addition, it is common to assume that the soils are homogeneous and isotropic. These assumptions make the analysis a lot simpler.

The specific gravity of the soil grains generally varies within a narrow range of 2.6–2.8. In other words, soil grains are 2.6–2.8 times heavier than water. With significant void content within the soil, where the voids are filled with air and/or water, the *bulk density* of soils can be in the range of 1.3−2.2 g/cm^3. The specific gravities of other materials are given in Table 3.3 for comparison.

3.5 Soil Classification

Soils can be classified into different groups such that the soils within a specific group show similar behavior. There are a few different ways of classifying the soil, depending on the purpose (for example, agricultural, roadwork, or geotechnical). There can also be slight differences between countries or regions. The *Unified Soil Classification System* (USCS) is the most common soil classification system used worldwide, for most geotechnical applications. The other country-specific soil

Specific gravity values for engineering materials and others

Table 3.3

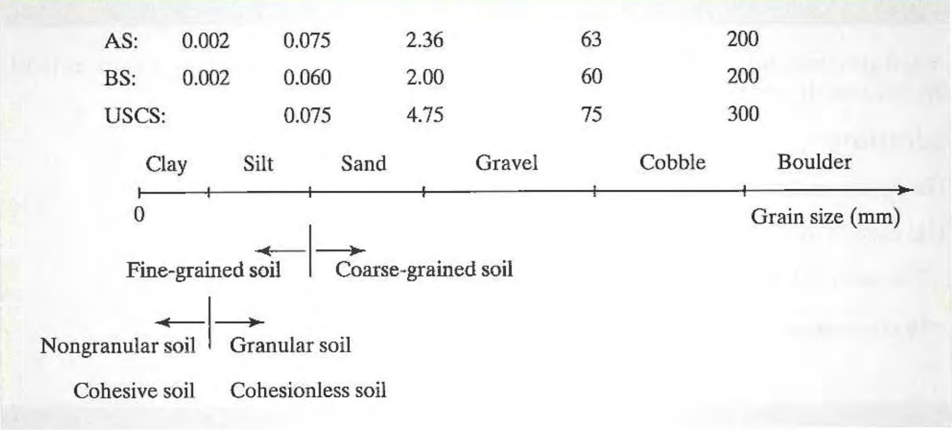
Material	Sp. gravity	Material	Sp. gravity
Wood	0.4–0.8	Soil grain	2.6–2.8
Gasoline fuel	0.72	Aluminum	2.6–2.8
Alcohol	0.80	Cast iron	6.8–7.8
Diesel fuel	0.8–0.9	Iron or steel	7.85
Kerosene	0.82	Stainless steel	7.5–8.0
Turpentine	0.87	Copper	8.9
Water	1.00	Silver	10.5
Asphalt	1.1	Lead	11.4
PVC	1.4	Mercury	13.6
Concrete	2.2–2.4	Gold	19.3
Porcelain	2.5	Platinum	21.4

classifications include Australian standards, British standards, Indian standards, and so forth. For roadwork, it is common to use the standards of the *American Association of State Highway Transportation Officials* (AASHTO), which are quite different from USCS.

Based on the grain size alone, the soils can be called boulders, cobbles, gravels, sands, silts, or clays. The size ranges for specific groups, based on the different standards, are summarized in Figure 3.5. What is often referred to as soils include mainly clays, silts, sands, and gravels. Cobbles and boulders have their place in civil engineering applications, such as earth- and rock-filled dams, road work, and so forth. While sands are called coarse-grained soils in a geotechnical context, they are treated as fine in concrete. The grains of a fine-grained soil such as silt or clay cannot be distinguished by a naked eye—it requires a microscope.

Major soil groups

Figure 3.5



Silts, sands, and gravels are known as *granular soils* or *cohesionless soils*. Clays are known as *nongranular soils* or *cohesive soils*, which can be sticky due to their cohesive nature. The grains of a granular soil are equidimensional, with similar dimensions in all directions. They can be angular, rounded, subangular, or subrounded. The clay particles are not equidimensional. They are more like flakes or needles, with very large surface area per unit mass.

The clay particles are negatively charged due to broken bonds and substitution of ions. The negative charge and the large surface area make them plastic and cohesive. Here, the surface forces have greater influence than the body forces. On the other hand, granular soils are inert with no charge imbalance, and the body forces are in control.

Granular soils are further classified based on their grain-size distribution. The engineering behavior of a granular soil is governed mainly by the grain-size distribution and how densely the grains are packed. The relative quantities of the different sizes of grains present in the soil are determined by sieving the soil through a stack of sieves.

In a cohesive soil, the grain-size distribution has very little influence on its engineering behavior. Here, the plasticity of the clay governs the engineering behavior. When water is added to a clay and mixed homogeneously, it can be molded between the fingers and rolled into a thread. The lowest water content at which the clay can be rolled into a 3 mm diameter thread is known as the *plastic limit*. *Water content* is simply the ratio of the mass of water to the mass of the dry soil, expressed generally as a percentage. When the water content is increased further, at one stage, the clay starts flowing like a liquid. This water content is known as the *liquid limit*. The clay behaves like a plastic material when the water content is between the plastic limit and the liquid limit. The difference between the liquid limit and the plastic limit is known as the *plasticity index*, which is a good measure of the plasticity of the clay. Clays are further classified on the basis of their liquid limit and plasticity index. The liquid limit and plasticity were originally introduced in the ceramics industry by a Swedish scientist Albert Atterberg in 1911 and hence are known as *Atterberg limits*.

Water content is the simplest measurement in soil mechanics. Drying the soil in an oven, at 105–110°C for 24 hours or more, drives out the moisture entirely. From the wet and dry masses of the soil, the water content can be determined.

EXAMPLE 3.1

A soil specimen taken from the ground weighs 395.2 g. When it is dried in an oven at 105°C for 24 hours, it weighs 364.8 g. What is the water content?

Solution:

The total mass, including the soil grains and water = 395.2 g

The mass of the dry soil grains = 364.8 g

$\therefore$ The mass of water in the soil = 395.2 − 364.8 = 30.4

$$\therefore \text{Water content} = \frac{30.4}{364.8} \times 100 = 8.3\%$$

3.6 Compaction and Earthworks

Very often, the soil conditions at a site do not meet the design requirement in the present form. The soil may be too weak, undergo excessive deformations, and/or lead to possible failure. Even if the soil at the surface is suitable, the subsoil conditions may be unfavorable. Designing the structure or facility to suit the existing soil conditions can be an expensive option. Instead, improving the ground is often preferred.

Compaction is a simple and inexpensive ground improvement technique that works on all types of soils. Some form of machinery, such as a mechanical roller, is used to compact and densify the soil, where water is added to the soil as a lubricant.

Earthmoving is a common name used for a group of activities in site preparation, which includes excavation, leveling, and compaction. This often requires heavy machinery such as excavators, dozers, graders, and so forth, shown in Figure 3.6. Figure 3.7 shows a site during and after the earthwork, along with the earthmoving equipment. Compaction is one of the major activities in the construction of earth- and rock-filled dams and highway embankments.

3.6.1 Moisture–Density Relationships

The compacted soil consists of all three phases, namely, the soil grains, water, and air. Water content is defined as the ratio of the mass of water to the mass of soil grains. Neglecting the mass of air, the total mass of soil (including water) is M_t, the mass of the soil grains M_s, and the water content w are related by

$$M_t = M_s\left(1 + \frac{w}{100}\right) \tag{3.1}$$

The bulk and dry densities (ρ_m and ρ_d) are computed, using the total mass of the soil (including water) and mass of the soil grains, respectively, and using the same volume. Therefore,

$$\rho_m = \rho_d\left(1 + \frac{w}{100}\right) \tag{3.2}$$

Figure 3.6

Some earthmoving machinery: (a) backhoe; (b) excavator; (c) spreader or dozer; (d) truck; (e) dump truck; and (f) roller

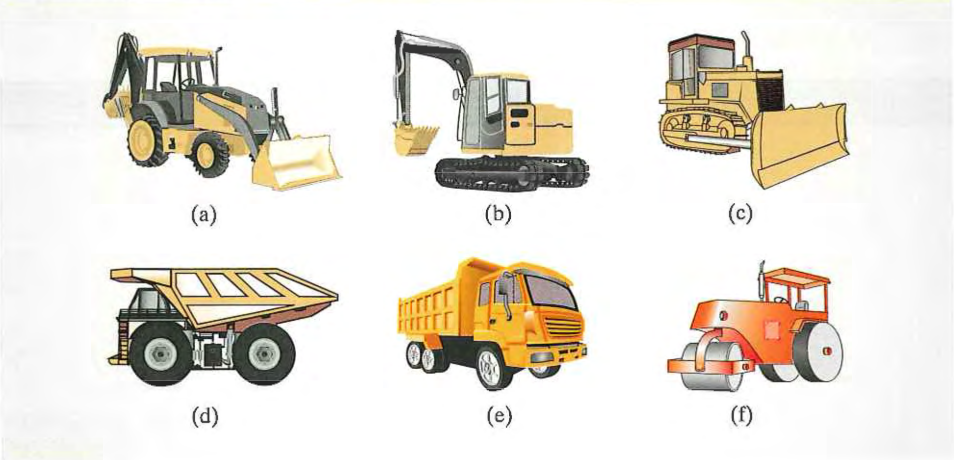


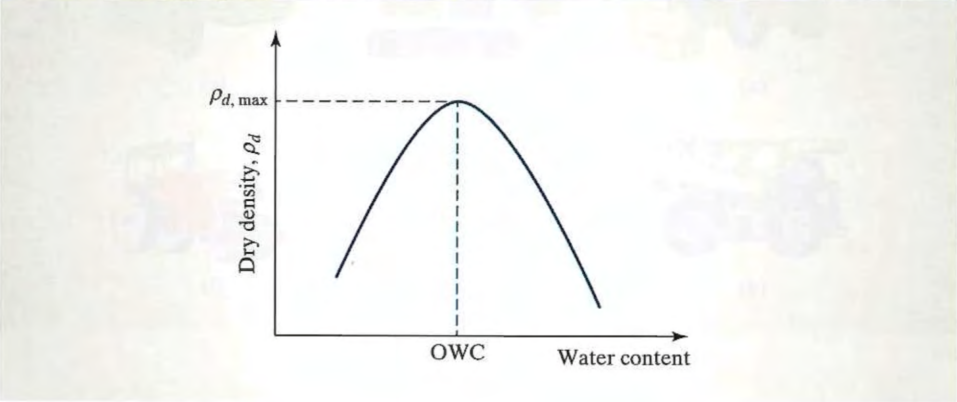
Figure 3.7 During and after the earthworks

(Courtesy of N. Sivakugan James Cook University, Australia)

The larger the dry density, the lower the void volume of the soil is. Therefore, dry density is a good measure of the effectiveness of compaction. It is also relatively easy to determine dry density from the bulk density and water content, using Eq. 3.2.

The water content of the compacted soil can affect the packing density of the soil grains and have significant influence on how the compacted earthwork behaves under loading (Proctor 1933a). Figure 3.8 shows the typical variation of dry density with water content for most soils. The dry density is the maximum ($\rho_{d,\max}$) at the *optimum water content* (OWC), where the void content is the lowest and the grains are packed very densely. The soils compacted at the optimum water content generally have the most desirable engineering properties, including high strength, high stiffness, and low permeability. Therefore, it is often necessary to know the optimum water content and the maximum dry density to plan the compaction work at the site. With increasing compaction effort, the compaction curves shift upward and to the left, with an increase in the maximum dry density and reduction in the optimum water content.

Figure 3.8 Compaction curve: moisture–density relationship



3.6.2 Laboratory Tests

In earthworks, the soil is generally placed in layers, known as *lifts*, and compacted. The lift thickness can be 100–500 mm, with cohesive soils at the lower end and granular soils at the higher end. To develop the compaction curve in the laboratory, the soil is placed in cylindrical molds and compacted in layers. The test was originally proposed by Proctor (1933b). The following example illustrates the interpretation of the compaction test data.

EXAMPLE 3.2

Six soil specimens were prepared from the same soil, compacted with a specific effort, in a rigid cylindrical mold with volume of 1000 ml. The total mass of the compacted specimens and their water contents are given below. Plot the compaction curve and determine the maximum dry density and the optimum water content.

Water content (%)	11.9	13.2	14.7	15.9	16.9	17.5
Specimen mass (g)	1746	1913	2053	2040	1905	1798

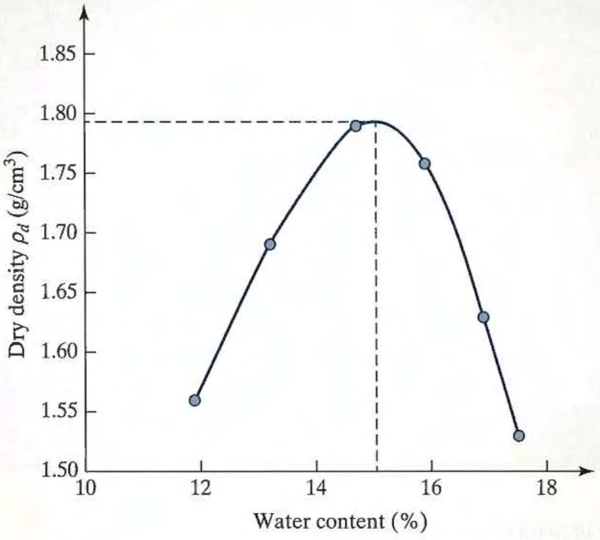
Solution:

Assuming a specimen volume of 1000 ml, the densities are computed as shown in the following table. The water content and dry density values are used in generating the compaction curve (Figure 3.9).

w (%)	M_t (g)	ρ_m (g/cm^3)	ρ_d (g/cm^3)
11.9	1746	1.746	1.560
13.2	1913	1.913	1.690
14.7	2053	2.053	1.790
15.9	2040	2.04	1.760
16.9	1905	1.905	1.630
17.5	1798	1.798	1.530

Compaction curve for data given in Example 3.2

Figure 3.9



3.6.3 Field Compaction, Specifications, and Control

There are different types of rollers to suit specific soil types. For example, vibrating rollers are effective on granular soils, and sheepsfoot rollers are effective on clayey soils by providing a kneading action. It is important to select the right type of roller and appropriate compactive effort for a specific job.

The engineering behavior of compacted clay soil is very sensitive to the water content. Depending on whether the compaction is carried out at water content slightly less or more than the optimum water content, the compacted earthwork can show significant differences in the swell-shrink behavior, strength, stiffness, and permeability of clays (Lambe 1958).

In field compaction, it is necessary to have stringent control over the water content and the dry densities achieved at the site. The compacted earthwork is checked at regular intervals by measuring the water content and the dry density using a nuclear density meter (Figure 3.10a) or sand replacement apparatus. In addition, there are other devices, such as the dynamic cone penetrometer (Figure 3.10b), Clegg hammer, and Proctor needle, that are used in the quality-assurance work on compacted earthwork. The dynamic cone penetrometer is an inexpensive and simple device used in roadwork and other compacted earthwork, where the resistance to penetration is correlated to strength and stiffness of the soil.

California bearing ratio (CBR) is commonly used as a measure of the strength of the compacted earthwork and the aggregate layers making the roadwork. Here, the

Figure 3.10 (a) Nuclear density meter and (b) dynamic cone penetrometer. Inset: falling weight and tip

(a) (b)

(Courtesy of N. Sivakugan, James Cook University, Australia)

resistance to penetration of a 49.6 mm diameter plunger, pushed into the compacted soil at a rate of 1.3 mm per minute, is compared to that of a reference material consisting of compacted crushed rock and expressed as a percentage. Fine-grained soils generally have CBR of less than 15. Densely compacted granular soils can have CBR of 20–80.

3.7 Permeability

Soils are particulate materials and hence porous media. The interconnected voids within the soil can act as pathways for facilitating the flow of water through soils. *Permeability* is a measure of how easily water can flow through a soil. The denser the soil, the lower the permeability is. *Hydraulic conductivity* is another name for permeability.

In geotechnical problems involving flow of water through geomaterials, permeability is an important parameter that has to be determined. This includes flow through earth- and rock-filled dams, flow beneath concrete dams, dewatering at excavations, ground water flow, and so forth.

3.7.1 Darcy's Law

According to Bernoulli's equation, the energy of a water molecule consists of three components, namely, potential energy, strain energy, and kinetic energy. This can be expressed as

$$\begin{aligned} \textit{Total energy} &= \textit{Potential energy} + \textit{Strain energy} + \textit{Kinetic energy} \\ &= mgz + \frac{mp}{\rho_w} + \frac{1}{2}mv^2 \end{aligned}$$

where m = mass, p = pressure, and v = velocity of the water molecule; g = gravity; and ρ_w = density of water.

$$\therefore \textit{Total energy per unit weight of the fluid molecule} = z + \frac{p}{\gamma_w} + \frac{v^2}{2g} \tag{3.3}$$

$$\textit{That is, Total head} = \textit{Elevation head} + \textit{Pressure head} + \textit{Velocity head}$$

where γ_w = unit weight of water (9.81 kN/m^3 or 62.4 lb/ft^3). All the energy terms in Eq. 3.3, known as "heads," have the unit of length. In flow through soils, the velocity of flow is so low that the velocity head is negligible.

Along the path of a water molecule, the total head (that is, energy) gradually decreases due to the work done in overcoming the frictional resistance provided by the soil. The total head reduction per unit length is known as the *hydraulic gradient*, a dimensionless quantity.

A French engineer Darcy (1956) noted that when the flow through soils is laminar, the velocity (v) is proportional to the hydraulic gradient (i). It can be written as

$$\begin{gathered} v \propto i \\ v = k\, i \end{gathered} \tag{3.4}$$

The proportionality constant k is called the permeability of the soil, which has the unit of velocity. Preferred units of permeability are cm/s and m/s.

3.7.2 Typical Values

In granular soils such as sands and gravels, permeability is related to the grain-size distribution and the porosity. In clays, it is governed by plasticity. Figure 3.11 gives typical values for the permeabilities of different soil groups. In gravels, the flow can be turbulent, and hence Darcy's law (Eq. 3.4) is not valid. Granular soils are generally well drained and are preferred as backfills in trenches and pits. Clays in general have low permeability, and high plastic clays can be almost impervious. They are used in making the clay cores of earth dams to minimize the seepage. Compacted clays with permeability of less than 10^{-7} cm/s are effective as liners at the bottom of the waste disposal containments in minimizing the leachates entering the ground water.

3.7.3 Laboratory and Field Tests

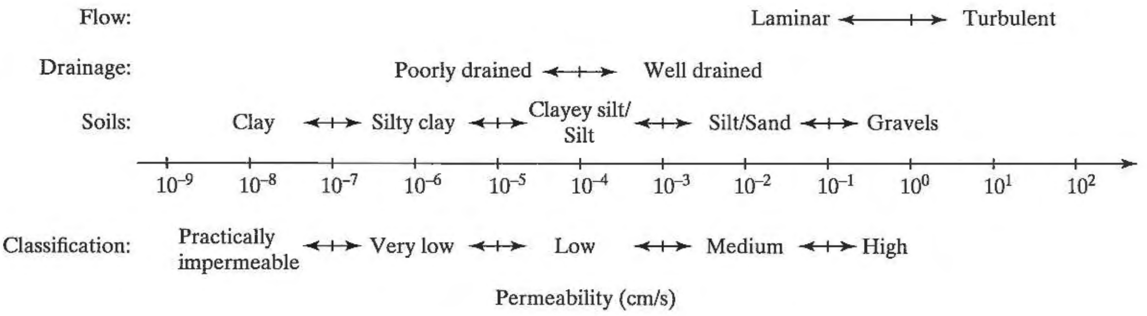
Permeability of a coarse-grained soil is generally determined through a *constant head permeability test* in the laboratory. Here, a cylindrical soil specimen is subjected to a constant total head over a period of 10–30 minutes, and the flow rate is measured. From the cross-sectional area of the specimen and the flow rate, the discharge velocity is computed. From the hydraulic gradient and the discharge velocity, permeability can be computed, using Darcy's law. In fine-grained soils, it can take several hours or days to collect appreciable quantity of water, and hence it is not possible to measure the flow rate. Here, permeability is determined through *falling head permeability tests*.

In the saturated zones in the field, permeability can be determined through pumping tests, where water is pumped in or out of a well and the rate of flow to maintain steady state is used in computing the permeability. In addition, there are borehole permeameters and infiltrometers that are commonly used for measuring permeability in the field, especially when the soil is not saturated. The presence of air within the soil matrix makes the flow difficult. Therefore, the permeability of an unsaturated soil is always less than that of saturated soil.

3.8 Strength and Stiffness

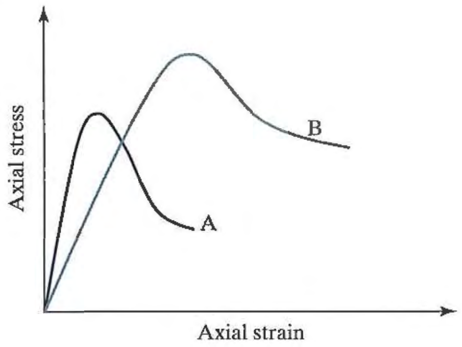
Strength and *stiffness* are two different measures governing the deformability of a material such as steel, concrete, or soil. Strength is the maximum stress that can be applied to the material before it fails, and it can be different in tension, compression,

Figure 3.11 Typical permeability values



Strength and stiffness of a material

Figure 3.12



or shear. The designer must know the strength so that the structure can be designed such that the stresses do not exceed the strength. Stiffness is a measure of how difficult it is to deform the material. It is opposite to compressibility. In a linear elastic material, Young's modulus (E), shear modulus (G), and bulk modulus (K) are all measures of stiffness (see section 1.4.1).

Figure 3.12 shows the stress-strain plots of two different materials, A and B. It can be seen that B has greater strength than A, and A has greater stiffness than B. Under typical loading conditions, where the stresses are well below the failure stresses, the deformations and strains in material A will be less than those in B, under the same stresses. Depending on the design requirement, both A and B have their place.

In *limit state design*, the structure or facility is designed to meet two performance criteria, namely, the *ultimate limit state* and *serviceability limit state*. The ultimate limit state requirements are that there should be no failure in any mode (for example, tension, compression, shear, and bending). The serviceability limit state requires that the structure or facility remains functional for its intended purpose and ensures that the deflection, settlement, vibration, and noise are within tolerable levels. Failure is related to *ultimate limit state*, and the deformations are related to the *serviceability limit state*.

3.8.1 Failure in Soils

Soil is a particulate medium, consisting of grains of different sizes and shapes packed together. The failure in soils generally takes place in shear, along a failure surface where the grains slide over each other. The shear stress at failure is known as the *shear strength*. The failure of individual grains under compressive or tensile stresses is rarely a concern, as it requires substantially larger stresses than those required to induce shear failure.

The shear strength (τ_f) and the normal stress (σ) along the failure surface are related by

$$\tau_f = \sigma \tan \phi + c \tag{3.5}$$

where ϕ = *friction angle*, and c = *cohesion* of the soil. The shear strength along the failure surface is derived from two independent components *friction* and *cohesion*.

The frictional component is proportional to the normal stress at the failure surface, and the cohesive component remains the same at any stress level. Granular soils, such as sands and gravels, are cohesionless soils (where $c = 0$) and hence their shear strength is purely frictional. When a sand is poured on a flat surface, it forms a conical pile. The angle between the conical surface and the horizontal surface is known as the *angle of repose*, which is about equal to the friction angle. The friction angle of granular soils typically is in the range of 30°–45°.

Eq 3.5 is often called *Mohr-Coulomb failure criterion*, or *Mohr-Coulomb failure envelope*, named after Charles-Augustin de Coulomb and Christian Otto Mohr. It is an elastic, perfectly plastic constitutive model. *Drucker-Prager* and *Cam Clay* are two sophisticated constitutive models that are also used to describe the behavior of geomaterials. When using software packages such as Plaxis, FLAC, or Abaqus for numerical modeling of soil-related problems, it is necessary to specify the constitutive model that would describe the soil the best. While the default is generally Mohr-Coulomb, the others, such as Drucker-Prager and Cam Clay, are also widely used for specific soils.

In traditional geotechnical engineering analysis, including foundations and retaining walls, Mohr-Coulomb is the preferred constitutive model. The two soil parameters required are cohesion c and friction angle ϕ. Tan ϕ can be seen as the frictional coefficient along the failure surface.

3.8.2 Deformations in Soils

In soils, deformations can cause as much problems as failures. The foundations of buildings are generally designed to settle less than 25–50 mm during their design life. The settlements within a structure can induce additional moments and stresses, which can crack the plaster, jam the partitions, and affect the alignments of service lines. When the settlements are excessive, the cracks can expose the reinforcements in the beam. Figure 3.13 shows the Leaning Tower of Pisa in Italy. After the recent restoration work, the 56 m high tower is still leaning 4° to the vertical. The top of the tower is displaced horizontally by about 3.9 m. The serviceability limit state is violated here, and the tower has become a world wonder and a major tourist attraction. In March 1990, the tower was closed to the public for safety reasons, but after some years of remedial measures, it was reopened to the public in 2001.

Deformations in soils are often computed, based on a linear elastic constitutive model, assuming Hooke's law. The Young's modulus and Poisson's ratio are the two main parameters for such elastic analysis of soils. The Young's modulus of soils can vary in the range of 2–100 MPa.

3.8.3 Effective Stress Theory and Drained/Undrained Loading

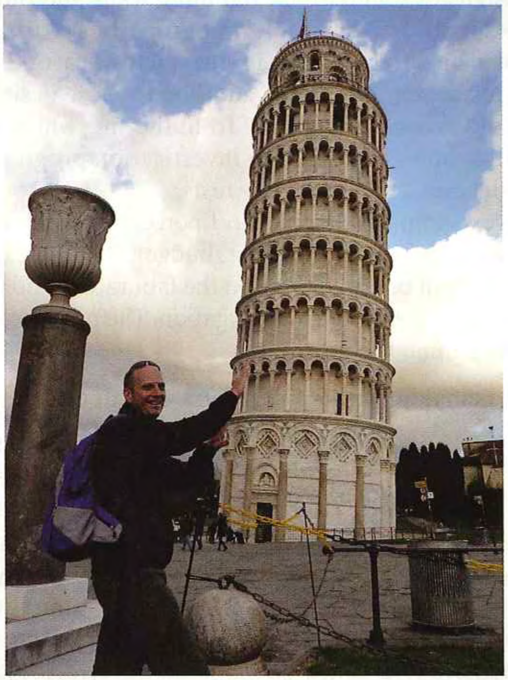
When a saturated soil element is subjected to external loads, the loads are shared by the soil skeleton (i.e., soil grains) and the water occupying the pores. Terzaghi's (1943) effective stress theory states that

$$\sigma = \sigma' + u \tag{3.6}$$

where σ = *total* normal stress, σ' = *effective* normal stress, the normal stress carried by the soil grains, and u = *pore water pressure*. Here, σ and σ' can vary with direction, whereas u is hydrostatic and is the same in all directions.

Leaning Tower of Pisa

Figure 3.13



(Courtesy of Dr. Briony Rankine, Golder Associates, Australia)

One of the topics in soil mechanics that you may find challenging or even confusing is the difference between drained and undrained loading and the choice of appropriate parameters for these two different loadings. This is particularly relevant to clay soils, which take months or years to drain the water completely when subjected to external loads. In the *shortterm*, immediately after the construction, hardly any drainage takes place. It is difficult to separate the loads carried by the soil grains and the soil skeleton, and hence the entire soil mass is treated as a whole and the analysis is carried out in terms of total stresses. In the long term, when the soil has fully drained and is without any excess pore water pressures, the analysis is generally carried out in terms of the effective stresses. Granular soils are free draining, and hence the analysis is always in terms of effective stresses. The strength and deformability parameters are different for drained and undrained loadings.

3.8.4 Laboratory and Field Measurements

Similar to other materials used in civil engineering applications, soil specimens are tested in the laboratory for determining the strength and deformation parameters. *Triaxial tests* and *direct shear tests* are the two most common tests carried out in commercial laboratories for this purpose. In triaxial tests, a 38–50 mm diameter specimen, with length/diameter ratio of 2, is subjected to different confining pressures and a vertical pressure, which is increased to failure. These data are used to develop the Mohr-Coulomb failure envelope given by Eq. 3.5, from which c and ϕ are determined. Field measurements of soil properties are discussed in section 3.9.2.

3.9 Measurements of Soil Properties

To carry out the designs of foundations, retaining walls and other earth-supported structures, it is necessary to know the relevant properties of the soils that are in the vicinity. These data can be obtained from in situ soil tests carried out at the site, using a wide range of soil testing equipment. In addition to these in situ tests, intact or disturbed soil samples are generally collected from the site, which can be tested in the laboratory. Figure 3.14 shows a simple site investigation program, which includes in situ tests, sample collection, and laboratory tests.

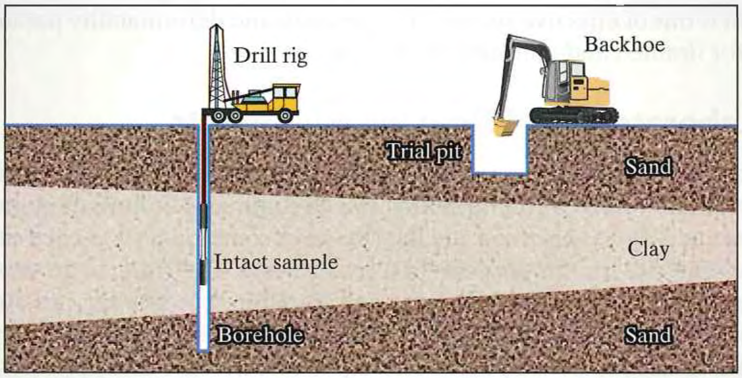
A typical site investigation can cost up to 1 percent of the total cost of the project, depending on the importance of the project, budget, and the extent of the exploration. The number and depth of boreholes and the laboratory and in situ tests required can add significantly to the cost of site investigation. The purpose of the site investigation program is to determine the following:

- The different layers making up the soil profile and thicknesses;
- The soil properties of the different layers;
- Water table depth; and
- Any harmful ions (for example, sulphates) present in the soil and pH.

Undisturbed soil samples are generally collected in 50–100 mm diameter metal tubes. In granular soils, such as sands, it is very difficult to obtain undisturbed samples for further laboratory tests—the samples would not be retained within the sampling tubes. There are special techniques such as freezing the ground and taking samples, but they can be expensive and hence uncommon. Therefore, in situ testing of sand is often the way to go in determining the design parameters. Sometimes, laboratory tests are carried out with a reconstituted sand specimen, where the grains are packed to a density comparable to that in situ.

Trial pits are limited to depths of about four meters. They are made using backhoes and are inexpensive. They enable visual inspection of the topsoil. To know the subsoil conditions at further depths, boreholes are required. The boreholes can be advanced to depths in excess of 100 meters, with samples collected at specific depths. Generally, one borehole is located for every 200–300 m^2. Boreholes are made using drill rigs and can add significantly to the cost of site investigation.

Figure 3.14 Site investigation



3.9.1 Laboratory Tests

Measuring the water content of a soil is fairly straightforward in the soils laboratory. From the specimen dimensions and specific gravity of the soil grains, the porosity and density can be determined. The grain-size distribution has significant influence on the behavior of granular soils but not clays. Plasticity is an important parameter in clays. It influences the strength, stiffness, and permeability of a clay soil.

A clay soil in situ has a fabric, developed over thousands of years of aging and cementation, which influences the soil behavior. It is desirable to test the soil so that the fabric is preserved, and hence the properties are more realistic. Inserting the drill rods and samplers into the boreholes can disturb the soil and break some of the bonds between the soil grains. Such mechanical disturbance can be minimized by using thin-walled sampling tubes. Undisturbed or intact samples are collected mainly in clay soils.

While undisturbed samples are required for testing the strength, stiffness, and permeability of the soil, disturbed samples are adequate for classification purposes, including grain-size distribution, Atterberg limit, and water content tests.

3.9.2 In Situ Tests

In situ testing means testing the soil at the site itself. They are carried out on granular soils and cohesive soils, using specialized equipment often mounted on vehicles such as trucks. Most common in situ tests involve a probe that is driven or pushed into the ground, and the resistance is measured. The soil resistance to penetration into the ground is used as a measure of strength and stiffness of the soil.

Sometimes, the access to the site can be a real problem, and getting the soil testing rig and the equipment in may require special arrangements (Figure 3.15a).

In situ testing: (a) mobilization; (b) in situ testing in progress. Figure 3.15

(a) (b)

(Courtesy of N. Sivakugan, James Cook University, Australia)

The *establishment* or *mobilization cost* can contribute significantly to the site investigation work. This is the cost associated with getting the soil testing equipment to the site and the initial preparations. Figure 3.15b shows a truck-mounted cone penetration test rig at work, where a drill rod with a cone at the end is pushed into the ground at a steady rate and the resistance is continuously measured.

Cone penetration tests and *standard penetration tests* are the two most commonly used in situ tests (Sivakugan and Das 2010). In a standard penetration test, a sampler at the end of the drill rod is driven into the ground by a 63.5 kg hammer. Vane shear tests, pressure meter tests, flat dilatometer tests, and plate loading tests are some of the other in situ tests carried out in soils.

3.9.3 Instrumentation

Using geotechnical instruments to monitor the performances of foundations, retaining walls, embankments, bridge abutments, and other structures is called *geotechnical instrumentation*. They are different from the measurements obtained in on-site investigation, where the one-off measurements are taken with the objective of interpreting the soil properties. In an instrumentation exercise, the measurements are ongoing for a certain period, and the objective is to measure deformations, strains, loads, pressures and temperatures to ensure they are within tolerable limits.

When working with new problems where the traditional theories and assumptions are questionable, it is often recommended to instrument the structure or facility and monitor the performance. This applies to new designs or construction methods. Records of instrumentation data can be powerful evidence in court cases and claims. They also can serve as early warning systems in mitigating potential disasters, such as landslides. They are very effective as design verification tools.

Strains are generally measured by strain gauges mounted at specific locations within the structure. Loads such as piles are measured by load cells, and the soil pressures are measured by earth pressure cells. Deformations are measured by dial gauges, extensometers, and inclinometers. The pore water pressure within the soil below the water table is measured by piezometers. Geotechnical instrumentation, the types of instruments, and their applications are discussed in detail by Dunnicliff (1993).

3.10 New Materials

Mine tailings, dredged spoils, and construction and demolition waste are some forms of waste materials that are available in large quantities worldwide and need to be disposed of. Here, mine tailings and dredged spoils are geomaterials, whereas construction and demolition waste can include crushed concrete, bricks, glass, and other materials.

Mine Tailings

The in situ rock has very low void content and density of 3–4 g/cm^3, depending on the mineralogy. During mining, the ore is removed from the ground from which the minerals are extracted, and a large quantity of crushed waste rock is present in the form of *tailings*. The mine tailings are literally soils, with grain sizes of clays, silts, or sands. When the tailings are placed underground to backfill the voids created during the mining, they are placed at dry densities of 1.5–1.8 g/cm^3 due to the voids between the grains. The volume of the tailings requiring disposal is at least 1.8 times the volume of the original in situ rock volume (Henderson and Revell 2005). A substantial quantity of the tailings that cannot be sent underground has to be disposed on the surface, through some tailings storage facilities such as tailing ponds.

Hydraulic fills and paste fills are two of the popular backfills used in underground mining. Hydraulic fills are sandy silts or silty sands with no clay content. The fine clay fraction is removed, in a process known as desliming, using hydrocyclones. This makes hydraulic fills behave as granular soils, where the traditional soil mechanics principles apply. Paste fill contains substantial clay fraction, and the grains are finer than hydraulic fills. Paste fills are often mixed with a small dosage (typically 2–5%) of binder such as ordinary portland cement, to enhance the strength on curing.

The crushing of the ore and the production of tailings takes place away from the underground voids. They are transported to the voids through pipes and boreholes in the form of slurry, at solid content of 75–80 percent by weight (Figure 3.16a). Rheological characteristics, including viscosity and yield stress, of the slurry play a key role in ensuring that the slurry can flow through the pipes without any blockages. The slurry consolidates and strengthens with time, enabling the nearby areas to be mined.

The underground voids can be approximated as rectangular prisms with base dimensions of 20–60 m and heights of 50–200 m, known as *stopes*. There are horizontal access drives at several levels, which are used by the trucks and other machinery. During filling, these access drives are barricaded (Figure 3.16b) so that the wet slurry is contained within the stope. Failure of these barricades can be catastrophic, and accidents have been reported worldwide.

Dredged Spoils

The waterways are regularly dredged to maintain adequate depths for the vessels. This produces millions of tonnes of sediments removed from the seabeds and riverbeds, which have to be disposed in a responsible manner. Unlike the mine fills, it

Figure 3.16

(a) Paste fill slurry pumped through pipe, and (b) a shotcreted 5 m × 5 m horizontal access drive barricade


(a) (b)

(Courtesy of N. Sivakugan, James Cook University, Australia)

is not possible to send the dredge spoils to where they came from. There are severe restrictions in place that limit or totally prohibit deep sea dumping. The sensible way of dealing with the dredge spoils is to use them in land reclamation, beach nourishment, and as a backfill material.

The methods commonly employed in dredging and land reclamation operations involve the use of cutter-suction dredgers, trailer-suction dredgers, grab dredgers, and hopper barges. Depending on the method employed, the dredge spoil can be called *ultra-soft clays* at high water content or *lumpy clays* at lower water contents. Figure 3.2 shows dredge spoils in the form of ultra-soft clays being pumped through a pipe into the containment paddocks.

When the dredge spoil is used in land reclamation as shown in Figure 3.2, the consolidation of the mud from very high water content and low strength to a firmer ground of adequate bearing capacity to support buildings and infrastructure can take years. Some ground improvement techniques are adopted to accelerate the consolidation process.

Construction and Demolition Waste

Millions of tonnes of solid wastes are generated every year by construction and demolition activities. They include concrete, brick, glass, steel, plastics, asphalt, and others. There have been attempts to recycle them by using them in roadwork and railway ballast. They can be used in gabion walls, which are made of wire cages filled with cobbles and smaller aggregates. When crushed, the construction and demolition waste materials are like granular aggregates and have the potential for use in many other engineering applications such as stone columns, backfilling, concrete, embankments, and so forth. There is always a strong competition from the virgin geomaterials, which are proven and have better properties. However, the lower cost and the environmental considerations give the recycled aggregates a chance.

Tire-Derived Aggregates

Every year, millions of recycled tires have to be disposed of in the United States and in other parts of the world. Tire-derived aggregates, such as tire shreds and tire chips, are being used in embankment and road construction, and as backfills (Bosscher et al. 1997). They have low unit weight, high permeability, and high insulation against frost penetration, with negligible effects when in contact with water. They are treated similar to geomaterials and are compacted and tested the same way.

3.11 Summary

1. More than 90 percent of earth's crust is made of silicates such as feldspar, quartz, mica, and the rest are nonsilicates, like carbonates, sulphides, oxides, and native elements.
2. Rocks can be classified as igneous, metamorphic, or sedimentary, depending on how they were formed. Similarly, soils are classified as residual or transported soils.
3. Soils are quite different from most engineering materials.
4. There are different soil classification systems for foundations, roadwork, airport pavements, and agricultural work.
5. Permeability is a measure of how easily water can flow through soils. The larger the permeability, more porous the soil skeleton is.

6. Young's modulus is a good measure of soil stiffness.
7. Shear strength of a soil is derived from two components, namely, friction and cohesion. Granular soils have no cohesion and their shear strength is purely frictional.
8. Soil properties are determined through a site investigation program, which consists of in situ tests, sampling, and laboratory tests.
9. Boreholes can cost a lot more than a trial pit. Boreholes are necessary for determining the soil conditions below about 4 m.
10. Instrumentation of foundations, retaining walls, bridge abutments, and embankments is carried out to monitor their ongoing performance through periodic measurements of strains, displacements, loads, and pressures.

Exercises

1. In descending order, the major elements of the earths crust are
   - **a.** O, Si, Fe, and Mg
   - **b.** Si, O, Fe, and Al
   - **c.** O, Si, Al, and Fe
   - **d.** O, Si, Mg, and Al
2. The thickness of the earths crust is
   - **a.** 1–5 km
   - **b.** 5–50 km
   - **c.** 50–500 km
   - **d.** More than 500 km
3. Nonsilicate mineral groups do not include
   - **a.** Carbonates
   - **b.** Sulfides
   - **c.** Quartz
   - **d.** Native elements
4. Which of the following is not an igneous rock?
   - **a.** Granite
   - **b.** Basalt
   - **c.** Gabbro
   - **d.** Marble
5. Which of the following is a sedimentary rock?
   - **a.** Limestone
   - **b.** Marble
   - **c.** Slate
   - **d.** Granite
6. Alluvial soils are transported by
   - **a.** Wind
   - **b.** Running water
   - **c.** Gravity
   - **d.** Ice

7. Which of the following is an organic soil?
    - a. Laterite
    - b. Peat
    - c. Loess
    - d. Marl

8. Which of the following is true about clays?
    - a. Having plasticity and cohesion
    - b. Having low permeability and hence poor drainage characteristics
    - c. Having the smallest grain sizes compared to other soil types
    - d. All the above

9. A likely value of Young's modulus of a soil is
    - a. 30 Pa
    - b. 30 kPa
    - c. 30 MPa
    - d. 30 GPa

10. Extensometers are used to measure
    - a. Loads
    - b. Displacements
    - c. Strains
    - d. Pressures

11. Which of the following soils have both frictional and cohesive components in the shear strength?
    - a. Sandy gravel
    - b. Gravelly sand
    - c. Silty sand
    - d. Sandy clay

12. The diameter of a borehole drilled during site investigation is
    - a. 50–75 mm
    - b. 200–300 mm
    - c. 500–1000 mm
    - d. Any of the above

13. State whether the following are true or false.
    - a. The continental crust is thicker than the oceanic crust.
    - b. Earth's crust is made mainly of silicates, rather than nonsilicates.
    - c. Silicon is the most abundant element in the earth's crust.
    - d. Igneous rocks are more abundant than metamorphic or sedimentary rocks in the earth's crust.
    - e. Slate is a sedimentary rock.
    - f. Marble is an igneous rock.
    - g. Permeability of a saturated soil is greater than that of the same soil when unsaturated.
    - h. In all soils, the shear strength is proportional to the normal stress on the shear plane.

14. What are the eight elements that make up the earth's crust? Which two of these are the most abundant?

15. List five nonsilicate rock-forming minerals.

16. What is the difference between magma and lava?

17. What are the two most common igneous rocks?

18. List five sedimentary rocks.

19. What are the major differences between soils and the other engineering materials?

20. What is the optimum water content in compaction?

21. What is hydraulic gradient?

22. A soil has cohesion of 10 kPa and friction angle of 26°. When shear failure takes place along a surface where the normal stress is 150 kPa, what is the shear strength of this plane?

23. Search the web and discuss the remedial measures taken on the Leaning Tower of Pisa to limit the settlement and tilt.

24. Write a short essay on geotechnical instrumentation, listing the different equipment used and the parameters measured. Include pictures as appropriate.

25. Surf the web and identify applications where tire-derived aggregates are used, and discuss the advantages.

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4

CHAPTER

The White Tower at the center of the Tower of London was built in the 11th century, using different types of rocks (e.g., limestones, mudstones). Gang Liu/Shutterstock.com

Rocks

4.1 Introduction

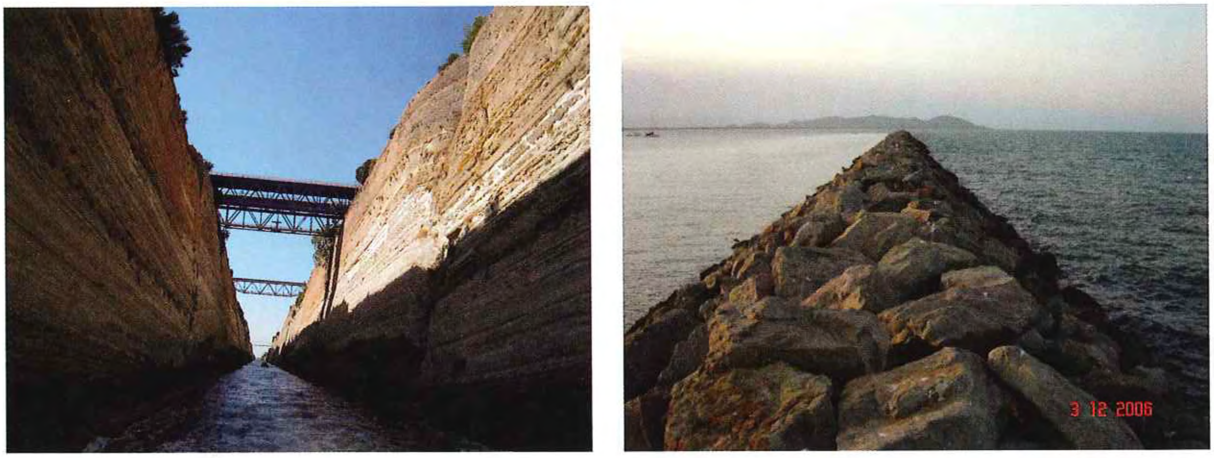
Rock is one of the most abundant materials near the earth's surface. Its low cost and abundance have made it a popular construction material since ancient times. It is widely used in buildings, dams, railway track beds, and road works. In addition, most civil engineering projects require working with rocks. These can be in the form of excavation through rocks, tunneling in rocks, foundations on rocks, underground mining, dams, blasting, and stability of rock slopes. Figure 4.1a shows the 6300-m-long Corinth Canal, connecting the Aegean Sea and Ionian Sea, cut through an overburden consisting of silty to clayey marls (cemented carbonate), overlain by weak sandstones and conglomerates. The canal was completed in 1893 after excavating 12 million cubic meters of ground. At some locations, the walls of the canals were as steep as 5:1, with a maximum depth of 79 m and average width at the bottom of about 21 m (Anagnostopoulos et al. 1991).

Figure 4.1b shows a breakwater with *armor stones* of about 1 m in size. Such large stones are provided in breakwaters, harbors, jetties, and coastal protection works to absorb the wave energy and prevent erosion. This is true for *riprap*, a layer of rocks placed over upstream faces of dams and shorelines to diffuse the wave energy.

Rocks are the basis of all geomaterials, including soils and aggregates. The geological processes of rock formation, the three categories of rocks, and some specific rock names were discussed briefly in Chapter 3. The rocks can be classified on the basis of their *geological origin, physical appearance*, or their *chemical composition*. On the basis of their geological origin, they can be classified as *igneous* (e.g., granite, basalt), *sedimentary* (e.g., sandstone, limestone) or *metamorphic* (e.g., marble, slate) rocks. Based on their physical appearance, they can be grouped as *stratified* (e.g., shale, sandstone) or *unstratified* (e.g., granite, marble) rocks. Based on their chemical composition, rocks can be classified as *siliceous rocks, argillaceous rocks*, or *calcareous rocks*. The siliceous rocks (e.g., granite, sandstone) have high silica content,

(a) Corinth Canal, Greece, (b) breakwater with armor stones

Figure 4.1



(a)

(b)

(Courtesy of N. Sivakugan, James Cook University, Australia)

argillaceous rocks (e.g., slate, laterite) are rich in aluminum, and calcareous rocks (e.g., marble, limestone) are rich in calcium carbonate.

Very often, for the sake of simplicity, engineering materials are assumed to be *c*ontinuous, *h*omogeneous, *i*sotropic, and *l*inear *e*lastic, which you can remember with the mnemonic "CHILE." The rock mass in nature defies all these assumptions; in reality, it is *d*iscontinuous, *i*nhomogeneous, *a*nisotropic, and *n*on-*e*lastic, which you can remember with "DIANE." This makes rock mechanics or rock engineering more challenging. This chapter gives an overview of rocks as engineering materials, their physical characteristics, and relevant laboratory tests for determining some parameters required in the designs and analysis.

4.2 Rock Engineering Applications

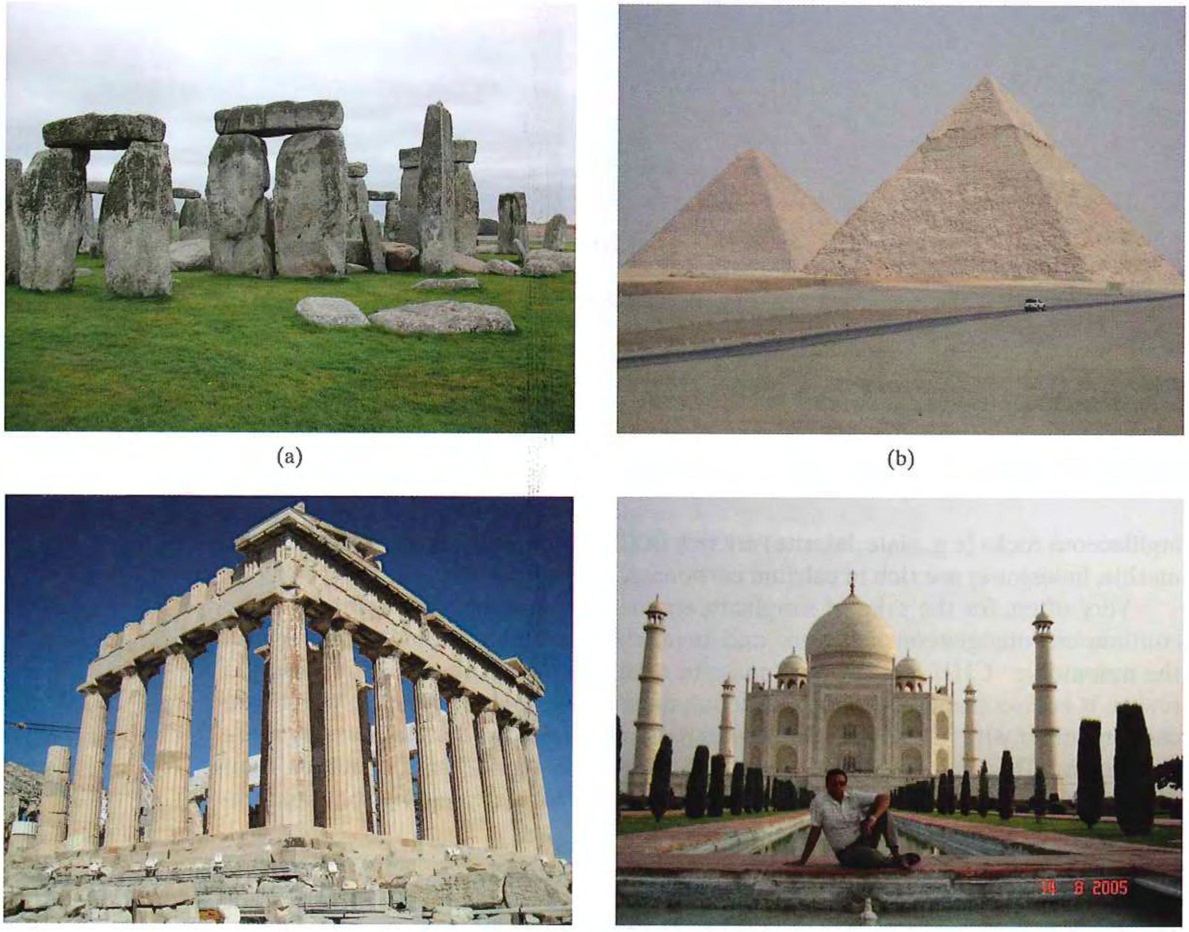
Rocks, and stones derived from rocks, have been used as building materials from the early days of civilization. Some examples include Stonehenge, the Egyptian pyramids, the Parthenon, and the Taj Mahal (Figure 4.2).

Stonehenge (Figure 4.2a) in the United Kingdom was built between 2500 and 2000 BCE, in several construction stages where the basaltic and sandstone stones were brought to the site from distant locations over a few centuries. The Egyptian pyramids (Figure 4.2b) were built around 2700 BCE in Egypt, using limestone and granite blocks. The Parthenon (Figure 4.2c) in Greece was built from 447–438 BCE out of marble, on top of the Acropolis hill consisting of limestone underlain by phyllites. The Taj Mahal (Figure 4.2d) in India was built in 1631–1653, using mainly marble and some sandstone.

4.2.1 Rocks as Construction Materials

Rocks are used as building materials in the form of *dimension stones*, as facing stones or claddings for floors and walls, and as structural stones for new buildings and restoration of old buildings. Dimension stones are natural rocks that are cut, drilled, or trimmed

Figure 4.2 Civil engineering marvels with rocks: (a) Stonehenge; (b) Egyptian pyramids; (c) Parthenon, (d) Taj Mahal



(a) (b)

(c) (d)

(Courtesy of N. Sivakugan, James Cook University, Australia)

to specific sizes and shapes. They are different from the fragmented stones produced by blasting where there is little control in their sizes and shapes. Granite, sandstone, limestone, marble, and slate are popular rock materials for making dimension stones. Restoration of many heritage-listed archaeological sites and monuments involves the use of dimension stones.

Rock-filled dams are made of compacted rockfills that contain very large aggregates, including cobbles and boulders. Rock-filled dams can be constructed relatively quickly and are economical when the rockfill material is available nearby. Gravel to cobble-sized (10–100 mm) aggregates are widely used as railway ballast, supporting the sleepers and spreading the dynamic loads from the railway carriages to the underlying ground. Roadwork is another application that uses large quantities of gravel and sand-sized aggregates in the base and subbase layers of the road. Durability of the aggregates under the repeated traffic loads is one of the main considerations in selecting the aggregates. Sand to gravel-sized aggregates are the main ingredients in concrete. Sands, gravels, cobbles, and boulders have been used for backfilling underground mine voids.

4.2.2 Other Civil Engineering Applications, using Rocks

In situ rock is a continuum that can span several meters, with or without the presence of joints at certain intervals. The rocks are removed from the ground through *blasting*, by the use of explosives. The blast design, including the choice of explosives, their quantities, the blasthole geometry, and the blasthole pattern, is influenced by the characteristics of the rock. Blasting is necessary in most of the excavation, tunneling, and mining work carried out in rocks. It is also required in quarries from which the stones and aggregates are removed. Blasting can induce vibrations to buildings in the region, which can lead to structural or architectural damage. In addition, the noise produced by blasting can be a source of complaints from the public. The ground vibrations and the noise travel long distances in the form of *body waves* (shear and compression waves) and *surface waves* (e.g., Rayleigh and Love waves). The vibrations and noise levels have to be monitored and ensured that they are within acceptable levels.

Underground transportation worldwide has relied extensively on tunnels made through soil and rocks hundreds of meters deep. For excavating tunnels through rocks, a tunnel-boring machine (TBM) is commonly used (Figure 4.3). Other excavations in rocks often require heavy machinery such as bulldozers, rippers, and hydraulic hammers, in addition to blasting.

Mining is a billion-dollar export industry in countries such as the United States, Canada, China, India, Brazil, Chile, Australia, and South Africa. *Open pit mining* and

Tunnel-boring machine

Figure 4.3

(Professor Michael Kavvadas, National Technical University of Athens) *underground mining* are two different ways of recovering shallow and deep ore bodies, respectively. The ore is removed through blasting and excavation, and the large voids created are backfilled with the crushed waste rock. The crushed waste rock can also be disposed on the surface through the use of *tailing dams*.

4.3 Common Rocks in Construction

Granite, basalt, sandstone, limestone, marble, and slate are the most common rocks used as construction materials.

Granite

Granite is a coarse-grained intrusive igneous rock, consisting of quartz, feldspar, and mica. It comes in pink to grey color. Classic granite has a "salt and pepper" look. Being one of the hardest rocks, it is very durable and can be polished to make it aesthetically appealing. It is least susceptible to scratches and retains its luster forever. It is used in luxury homes and offices, in kitchens, bench tops, counter tops, and floor surfacing.

Basalt

Basalt is one of the most abundant rocks found on the surface of earth. It is a hard and dense, fine-grained, extrusive igneous rock. Most of the oceanic crust is made of basalt; most of the continental crust is granite. Basalt has high resistance to heat and fire. It is used for sculpture, railway ballast, and as aggregates for roadwork and concrete. A most promising application of basalt is in the form of fine fiber, produced by extrusion of basalt, where it has many desirable features such as insulation from heat and sound, vibration resistance, and durability. Like granites, they can be used for bench tops, fireplaces, wall panels, interior decoration, and decorative stones.

Sandstone

Sandstone is a sedimentary rock, formed by cemented sand grains, with quartz and feldspar as the major minerals. It was a popular building material in the ancient times, used extensively in buildings, temples, cathedrals, and monuments. In modern times, it is used on floors, walls, and as pavers. It comes generally in shades of brown, yellow, tan, and grey. It is relatively soft and hence easy to carve and work with. It is quite porous for a rock. It has good resistance against weathering. Its resistance to acids and alkalis, resistance to air and saline water, resistance to corrosion and weathering, very good binding with cement, load bearing, and heat resistance make it ideal for monuments.

Limestone

Limestone is a sedimentary rock, made primarily of calcium carbonate ($CaCO_3$). About 10 percent of the sedimentary rocks are limestone. It is easy to cut into blocks and slabs for use as a building material. In addition to be used as a building material and aggregate for road work, it is also crushed and used as a raw material in the production of cement, *quicklime*, and *slaked lime* and sometimes in glass, too. It has been a popular building material for its abundance near the earth's surface and its durability. From the time of the pyramids to modern times, limestone has been used in many buildings, churches, and castles. It was used in the Empire State Building, New York; St. Paul's Cathedral, London; and the Pyramid of Giza.

Over a very long time, limestone formations gets partially dissolved in percolating water that can be slightly acidic and develop into sinkholes, caverns, and caves, forming karst topography. These regions can undergo sudden settlements due to collapse of the overburden into these voids.

When limestone is heated, it produces quicklime (CaO). The chemical reaction is given by

$$CaCO_3 + \text{heat} = CaO + CO_2$$

A good way of identifying limestone is to add a drop of acid, which would produce a fizz due to the release of CO_2. Quicklime reacts with water, forming slaked lime [$Ca(OH)_2$], which is a good mortar for holding bricks, blocks, and stones together. The chemical reaction can be written as

$$CaO + H_2O = Ca(OH)_2$$

Marble

Marble is a metamorphic rock, derived from limestone. It is denser and harder than limestone. It is mostly white, but also comes in shades of pink, red, grey, green, brown, and black. The three major uses of marble are construction, interior decoration, and sculpture. The Taj Mahal and the Parthenon are two examples where marble was used extensively in construction. Michelangelo's *David* (1501–1504) and many ancient Greek and Roman sculptures are made of marble. Marble is used in kitchens, bathrooms, stairways, floors, and walls. It has high heat and fire resistance.

Slate

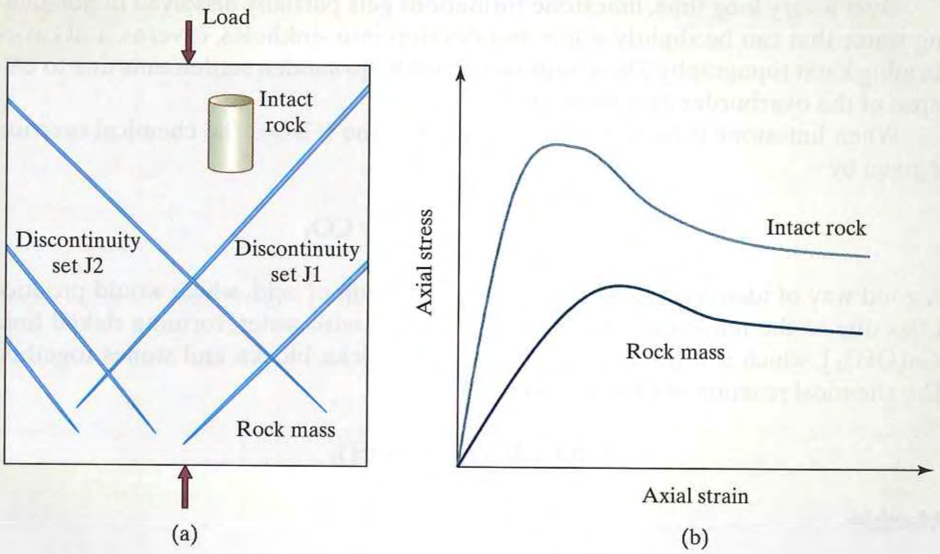
Slate is a metamorphic rock from clays and volcanic ash and is used for roof tiles, flooring, and pavers. It comes in shades of grey, but other colors are possible, too.

4.4 Rock Mass and Intact Rock

Rock is a continuum that extends from several meters to few kilometers. *Rock mass* is the term used for this large extent of rock which may have some *discontinuities* as well. The discontinuities include joints, faults, and bedding planes. Figure 4.4a shows the schematic diagram of a rock mass with two *sets of discontinuities* that are almost perpendicular to each other. A set of discontinuity often contains more than one joint, fault, or bedding plane, but they are approximately parallel. Presence of discontinuities can fragment the rock mass and make it weaker.

Discontinuity is a generic term used to describe a fault, joint, bedding plane, foliation, cleavage, or schistosity. A *fault* is a planar fracture along which noticeable movement has taken place. *Joints* are filled or unfilled fractures within the rock mass that do not show any sign of relative movement. *Bedding planes* are formed when the sediments are deposited in the rock formation process, creating planes of weakness, which are not necessarily horizontal. They are common in sedimentary rocks. *Foliation* occurs in metamorphic rocks where the rock-forming minerals exhibit platy

Figure 4.4 Rock mass and intact rock: (a) schematic diagram, (b) stress-strain curves



structure or banding, thus developing planes of weakness. *Cleavages* are planes of weaknesses that occur often as parallel layers and are formed in a metamorphic process. *Schistosity* is a type of cleavage seen in metamorphic rocks such as schists and phyllites, where the rocks tend to split along parallel planes of weakness.

When cylindrical rock cores are collected from boreholes during a site investigation exercise, they often break along the discontinuities and what we test is the *intact rock*, which tells nothing about the discontinuities. The load-deformation behavior of the rock mass under a specific loading condition (for example, foundation or tunneling) can be very different from that of the intact rock specimen, thanks to the discontinuities. Due to the presence of discontinuities, the rock mass is weaker than the intact rock specimen, showing lower strength and stiffness (see Figure 4.4b). In addition, the rock mass is more permeable, with the discontinuities allowing greater access to water, which can make the rock mass even weaker. Water reduces the friction along the discontinuities, and the increased pore water pressure reduces the effective stresses and hence the shear strength.

The rock cores are intact rock specimens, which are so small that they are often free of discontinuities. Even when they break along discontinuities, we trim them further to have a joint-free cores for the laboratory tests. On the other hand, the larger rock mass may have one or more sets of discontinuities that can have significant influence on the overall stability, which is not reflected on the intact rock specimen. Therefore, engineers have to be careful when using the data from intact rock to infer the behavior of the rock mass.

4.5 Strength and Stiffness of Intact Rocks

When discontinuities are present, the behavior of the rock mass is often governed more by the characteristics of discontinuities (e.g., orientation, spacing, width) than the properties of the intact rock. The high strength of the intact rock may not matter much when the failure takes place along the discontinuity. However, for practical reasons, it is difficult to carry out any laboratory tests on large specimens comprising the intact rock and discontinuities to assess the behavior of the rock mass. Therefore, it is a common practice to test intact cylindrical rock cores recovered from the ground through boreholes and use them as the basis, along with other information about the discontinuities, to infer the behavior of the rock mass. In addition, there are in situ tests that are carried out at the site, which would reflect the rock mass behavior better.

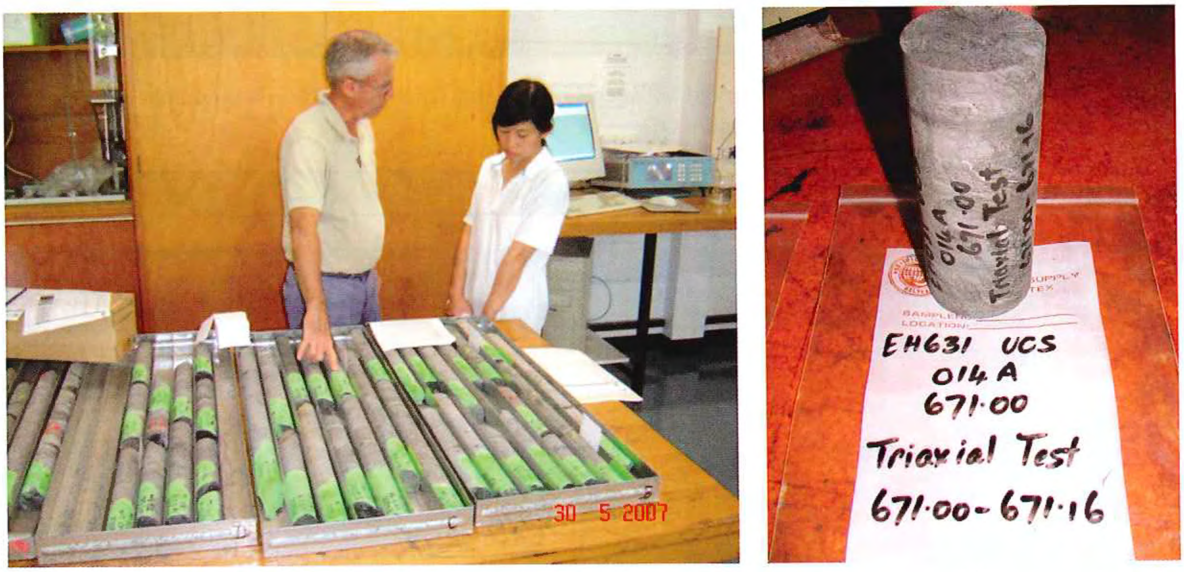
4.5.1 Rock Cores and RQD

Due to the high strength of rock, it is necessary to use thick-walled core barrels with tips made of hard materials, such as diamond or tungsten carbide, for coring into the parent rock. The rotary drill grinds away an annular zone around the rock core, which is collected into a 0.5–3.0 m long core barrel. The rock cores are labeled, showing the borehole number and elevation, and placed in a sample tray for transporting to the laboratory for testing (Figure 4.5a). The cores are cut into appropriate lengths, and the ends are flattened and polished before they are tested in the laboratory. Figure 4.5b shows a specimen prepared for a uniaxial compression test. Some of the common core sizes and their standard designations are given in Table 4.1.

Larger cores give better representation of the intact rock. However, the cost of sampling increases significantly with the core diameter. For most laboratory tests, it is suggested that the cores should be at least the size of NX cores. When coring in a

Rock cores: (a) cores from a site in a sample tray, (b) a rock specimen

Figure 4.5



(a) (b)

(Courtesy of N. Sivakugan, James Cook University, Australia)

Table 4.1 Rock core sizes and designations

Symbol	Nominal *core* diameter		Nominal *hole* diameter	
	(mm)	(in.)	(mm)	(in.)
AQ	27.0	1–1/16	48.0	1–57/64
BQ	36.5	1–7/16	60.0	2–23/64
NQ	47.6	1–7/8	75.8	2–63/64
HQ	63.5	2–1/2	96.0	3–25/32
PQ	85.0	3–11/32	122.6	4–53/64
EX	22.2	7/8	36.5	1–7/16
AX	30.2	1–3/16	47.6	1–7/8
BX	41.3	1–5/8	58.7	2–5/16
NX	54.0	2–1/8	74.6	2–15/16

rocky terrain, the core is broken at places where there are discontinuities, and some segments may be "lost" in the borehole. Core recovery ratio (CR) is defined as:

$$\mathrm{CR}(\%) = \frac{\text{Length of rock core recovered}}{\text{Total length of the core run}} \times 100 \tag{4.1}$$

Rock quality designation (RQD) was defined by Deere (1964) as

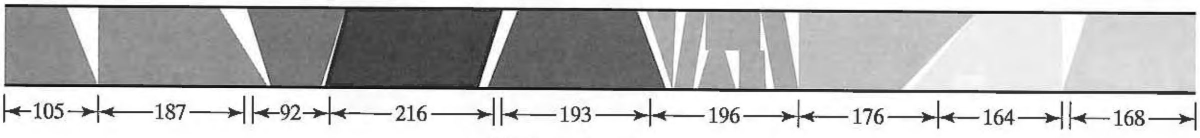
$$\mathrm{RQD}(\%) = \frac{\Sigma\ \text{Length of core pieces at least 100 mm in length}}{\text{Total length of the core run}} \times 100 \tag{4.2}$$

These two parameters CR and RQD are simple measures of the quality of the rock mass. The lower the RQD, more fractured the rock mass is.

EXAMPLE 4.1

The rock cores recovered between 123.0 m and 125.0 m depths in a bore hole are shown in Figure 4.6. Find CR and RQD.

Figure 4.6 Rock cores for Example 4.1



All dimensions in mm

Solution:

$$\mathrm{CR}(\%) = \frac{105 + 187 + 92 + 216 + 193 + 196 + 176 + 164 + 168}{2000} \times 100 = 74.9\%$$

$$\mathrm{RQD}(\%) = \frac{105 + 187 + 216 + 193 + 176 + 164 + 168}{2000} \times 100 = 60.5\%$$

Table 4.2 Rock quality designations and allowable bearing pressures

RQD (%)	Rock quality	Allowable bearing pressure (MPa or ton/ft²)
0–25	Very poor	1–3 (10.4–31.3)
25–50	Poor	3–6.5 (31.3–67.9)
50–75	Fair	6.5–12 (67.9–125.3)
75–90	Good	12–20 (125.3–208.9)
90–100	Excellent	20–30 (208.9–313.3)

Note: ton/ft² values in parentheses

RQDs and corresponding classifications of in situ rock quality are given in Table 4.2. Also given in this table are the approximate allowable bearing pressures for foundations on rock, as suggested by Peck et al. (1974).

4.5.2 Strength and Stiffness Parameters

Uniaxial compressive strength is the most used parameter in rock designs. It is the same as *unconfined compressive strength*. It is denoted by σ_c or UCS. It is the maximum normal stress a cylindrical specimen can carry before failure when it is unconfined. From the stress-strain curve plotted during the uniaxial compression test, the Young's modulus (E) can be determined.

Unlike soils, intact rocks have significant *tensile strength* (σ_t). Neglecting the tensile strength can result in conservative designs. The tensile strength of an intact rock is one of the key parameters in the design and numerical modeling of structures in rocks. At low-stress levels, for simplicity, rocks can be assumed to behave as *linear elastic materials*. Here, any analysis of deformations require Poisson's ratio (ν), which varies in the range of 0.1–0.4 for rocks. Some typical values of different rock types are summarized in Table 4.3.

With little more sophistication, the rock can be modeled as a Mohr-Coulomb material (see Section 3.8.1), where the two parameters required are cohesion (c) and friction angle (ϕ).

Uniaxial Compressive Strength

Clay soils have uniaxial compressive strength σ_c and Young's modulus E, quoted in kPa and MPa (or lb/ft² and ton/ft²), respectively. In rocks, they are orders of magnitude greater and are given in MPa and GPa (or ton/ft²), respectively. How do you differentiate hard soils (for example, hardpan) and soft rocks (for example, shale)? The commonly used cutoff that separates soils and rocks is uniaxial compressive strength of 1 MPa (10.4 ton/ft²); anything above this value is a rock. Figure 4.7 shows possible ranges in σ_c values for some common rock types. Note that the uniaxial compressive strength of concrete is typically 30–40 MPa (313.3–417.7 ton/ft²).

Stiffness

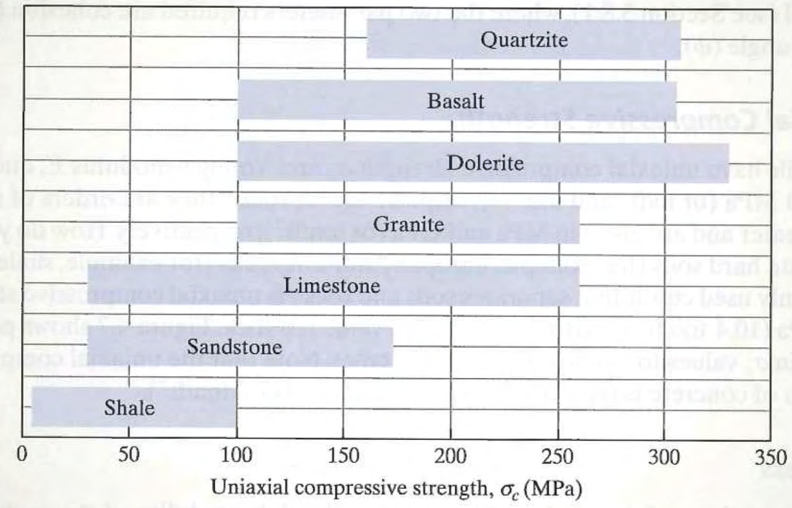
Stiffness of the rock is required for assessing the deformability of the rock. From the modulus of the intact rock and necessary details about the discontinuities, the modulus of the rock mass can be estimated. By measuring the axial deformations

Table 4.3 Typical values of Poisson's ratio for rocks (Gercek 2007)

Rock type	ν
Andesite	0.20–0.35
Basalt	0.10–0.35
Conglomerate	0.10–0.40
Diabase	0.10–0.28
Diorite	0.20–0.30
Dolerite	0.15–0.35
Dolomite	0.10–0.35
Gneiss	0.10–0.30
Granite	0.10–0.33
Granodiorite	0.15–0.25
Greywacke	0.08–0.23
Limestone	0.10–0.33
Marble	0.15–0.30
Marl	0.13–0.33
Norite	0.20–0.25
Quartzite	0.10–0.33
Rock salt	0.05–0.30
Sandstone	0.05–0.40
Shale	0.05–0.32
Siltstone	0.05–0.35
Tuff	0.10–0.28

(Based on Gercek, H. (2007). "Poisson's ratio values for rocks." International Journal of Rock Mechanics and Mining Sciences, 44(1), 1–13.)

Figure 4.7 Some typical σ_c values (Adapted from Hudson 1989)



(Based on Hudson, J.A. (1989). Rock mechanics principles in engineering practice, Butterworths, London.)

during a uniaxial compression test on an intact rock specimen, its stress-strain plot can be generated. From the stress-strain plot, σ_c and E can be determined. An E/σ_c ratio, known as *modulus ratio*, varies in a narrow range for a specific rock type. Table 4.4 shows the typical values of modulus ratios for different rock types. The modulus ratio of concrete is about 1000.

Shear modulus or modulus of rigidity (G) and bulk modulus (K) are sometimes required in the analysis. They were discussed in Section 1.4.1 and are related to Young's modulus and Poisson's ratio through Eqs. 1.14 and 1.19.

The axial strain at which the specimen fails is a measure of the ductility of the rock. Table 4.5 presents a simple classification suggested by Handin (1966). Ductile rocks show strain-softening behavior (Figure 1.5) undergoing large deformations before failure. This gives some early warning before any imminent collapse. Brittle rocks fail at low strains and show little or no warning before any collapse.

Modulus ratios for different rock types (after Hoek and Diederichs 2006) Table 4.4

	Texture			
	Coarse	**Medium**	**Fine**	**Very fine**
Sedimentary	Conglomerates 300–400	Sandstones 200–350	Siltstones 350–400	Claystones 200–300
	Breccias 230–350		Greywackes 350	Shales 150–250*
				Marls 150–200
	Crystalline limestone 400–600	Sparitic limestone 600–800	Micritic limestone 800–1000	Dolomite 350–500
		Gypsum (350)**	Anhydrite (350)**	Chalk 1000^{+}
Metamorphic	Marble 700–1000	Hornfels 400–700	Quartzite 300–450	
		Metasandstone 200–300		
	Migamatite 350–400	Amphibiolites 400–500	Gneiss 300–750*	
		Schists 250–1100*	Phyllites/Mica Schist 300–800*	Slates 400–600*
Igneous	Granite^{+} 300–550	Diorite^{+} 300–350		
	Granodiorite 400–450			
	Gabro 400–500	Dolerite 300–400		
	Norite 350–400			
	Porphyries (400)**		Diabase 300–350	Peridotite 250–300
		Rhyolite 300–500	Dacite 350–450	
		Andesite 300–500	Basalt 250–450	
	Agglomerate 400–600	Volcanic Breccia (500)**	Tuff 200–400	

* *Highly anisotropic rocks: the modulus ratio is significantly different if normal strain and/or loading occurs parallel (high modulus ratio) or perpendicular (low modulus ratio) to a weakness plane. Uniaxial test loading direction should be equivalent to field application.*

$^{+}$ *Felsic Granitoids: coarse-grained or altered (high modulus ratio), fine-grained (low modulus ratio)*

** *No data available; estimated on the basis of geological logic*

(Based on Hoek, E. and Diederichs, M.S. (2006). "Empirical estimation of rock mass modulus." International Journal of Rock Mechanics and Mining Sciences, Elsevier, 43(2), 203-215.)

Table 4.5 Classification of rocks based on failure strains (Handin 1966)

Classification	Axial strain (%)
Very brittle	<1
Brittle	1–5
Moderately brittle[1] (transitional)	2–8
Moderately ductile	5–10
Ductile	>10

[1] *Note the overlap*

(Based on Handin, J. (1966). "Strength and ductility." Handbook of Physical Contacts, Ed. S. P. Clark, Geological Society of America, New York, 223–289.)

EXAMPLE 4.2

For preliminary designs, estimate the shear and bulk modulus of an intact sandstone rock specimen, based on typical values.

Solution:

From Figure 4.7, $\sigma_c \approx 100$ MPa

From Table 4.4, modulus ratio ≈ 275

Therefore, $E = 27500$ MPa $= 27.5$ GPa

From Table 4.3, $\nu = 0.2$

Substituting in Eq. 1.14,

$$G = \frac{E}{2(1+\nu)} = \frac{27.5}{2(1+0.2)} = 11.5 \text{ GPa}$$

Substituting in Eq. 1.19,

$$K = \frac{E}{3(1-2\nu)} = \frac{27.5}{3(1-2\times0.2)} = 15.3 \text{ MPa}$$

Wave Velocities and Rock Properties

When an earthquake or blast occurs, a large amount of energy is released, which is propagated through surrounding soils and rocks in the form of *body waves* and *surface waves*. Body waves travel though the body of the material; surface waves travel along the surface of the material or along the interface between two materials. The two major types of body waves are *compression waves* and *shear waves*. Compression waves travel faster than shear waves, and hence would be recorded first when they are monitored after an event such as an earthquake or blast. Therefore, a compression wave is known as a *primary wave* (or P-wave) and a shear wave as the *secondary wave* (or S-wave). A compression wave is also known as a longitudinal wave or pressure wave. Here the direction of vibration (that is, particle velocity) is the same as the direction of propagation. In the case of shear waves, the direction of vibration is perpendicular to the direction of propagation. P-waves can travel through gases, liquids, and solids; S-waves travel only through solids (Liquid or gas cannot carry shear stresses). P-waves travel in the air as sound waves, at a velocity of 330 m/s (1080 ft/s). They travel at 1450 m/s (4760 ft/s) in water, 3600 m/s (11,810 ft/s) in concrete, and approximately 5000 m/s (16,400 ft/s) in granite. In rocks, S-waves travel at approximately 50 to 60 percent of the P-wave velocities. In general, the seismic waves travel faster in denser materials, and the velocities in soils and rocks vary in the range of 500–6000 m/s (1640–19,685 ft/s).

Surface waves are slower than the body waves. *Rayleigh waves* and *Love waves* are surface waves. The surface waves attenuate when they get deeper into the body of the material. They contribute significantly toward structural damage during earthquakes.

Generally, the wave velocities are greater in rocks than in water or air. Table 4.6 shows the compression and shear wave velocities through different rock types. In spite of the wide range of possible velocities and their overlaps between rock types, you can infer the underlying rock types from the wave velocities. There are several techniques for measuring the wave velocities at a site or from soil or rock specimens in the laboratory. From the shear or compression wave velocities, the soil or rock modulus can be determined through the following equations:

$$v_p = \sqrt{\frac{K + \frac{4}{3}G}{\rho}} \tag{4.3}$$

and

$$v_s = \sqrt{\frac{G}{\rho}} \tag{4.4}$$

v_p = P-wave velocity, v_s = S-wave velocity, K = bulk modulus, G = shear modulus, and ρ = density. In Eq. 4.4, $G = 0$ for gases (such as air) and liquids (such as water) that cannot carry shear stresses, which explains why gases and liquids cannot transmit S-waves. From Eqs. 4.3 and 4.4 and Table 4.6, it can be seen that v_p is always greater than v_s, and the ratio v_p/v_s can be in the range of 1.5–4.0.

By measuring the wave velocities through the ground, it is possible to identify the soil and rock types and to define the profile consisting of different layers. In addition, there are laboratory techniques for measuring the properties of a rock specimen by sending seismic waves through the specimen and measuring the velocities.

Porosity is defined the same way in rocks as in soils. It is the percentage of the volume occupied by the voids. Porosity of a rock can be less than 1 percent in granites and as much as 30 percent in shales and porous sandstones. The density of a rock typically varies in the range of 2500–2900 kg/m^3. It depends on the mineralogy and the porosity. Igneous rocks like granite and basalt and metamorphic rocks like marble are at the upper end of the scale, and sedimentary rocks such as sandstone and shale are at the lower end.

Mohs' Hardness Scale

In 1812, Friedrich Mohs (1773–1839), an Austrian mineralogist, proposed a hardness scale for minerals that is still used for quantifying the hardness of minerals, rocks, and metals. A mineral is scratched with another mineral to determine which of the

Table 4.6 Typical wave velocities in different rock types

Material	v_p(m/s)	v_s(m/s)
Granite	5000–6000 (16,400–19,685)	2800–3500 (9,190–11,480)
Basalt	5000–6000 (16,400–19,685)	2800–3500 (9,190–11,480)
Sandstone	1500–4000 (4,920–13,120)	750–2500 (2,460–8,200)
Limestone	5000–6000 (16,400–19,685)	2800–3500 (9,190–11480)
Saturated sand	800–2000 (2,625–6,560)	400–1000 (1,310–3,280)
Clay	1000–2500 (3,280–8,200)	400–1000 (1,310–3,280)
Dry sand	500–1200 (1,640–39,400)	200–500 (655–1,640)

Note: values in parentheses are in ft/s.

two is harder. A harder mineral scratches a softer mineral and not vice versa. If they scratch each other, they both have the same hardness. The scale is not linear. The softest material, talc, was assigned hardness number of one, and the hardest material, diamond, was assigned ten, with all others lying in between these two extremes. Table 4.7 summarizes the hardness numbers for the ten commonly available reference

Table 4.7 Mohs' relative hardness scale

Hardness Number	Mineral or Material
1	**Talc**, piece of chalk
1.5	Lead, graphite, tin
2	**Gypsum**, plaster of Paris
2.5	Finger nail, pure gold, silver, aluminum, galena, pearl
3	**Calcite**, ivory, copper, limestone
3.5	Copper coin (penny)
4	**Flourite**, coral, nickel
4.5	Wire nail
5	**Apatite**, asbestos
5.5	Window glass, steel knife
6	**Orthoclase feldspar**, titanium, stainless steel, hematite
6.5	Steel file
7	**Quartz**, tourmaline, ceramic tile
7.5	Garnet, tourmaline
8	**Topaz**, hardened steel, emerald, basalt
8.5	Chromium
9	**Corundum**, ruby
9.5	Carborundum, tungsten carbide
10	**Diamond**

minerals originally proposed by Mohs (in bold) along with some other commonly known materials. Granite has hardness number of 5–7, and marble has 3–5. Rocks contain several different minerals, and due to heterogeneity and the variability in the composition, most rock types do not have a unique hardness value; they lie in a narrow range.

4.6 Laboratory Tests for Intact Rocks

The common laboratory tests on cylindrical intact rock cores recovered from boreholes include the uniaxial compression test, indirect tensile strength test, point load test, Schmidt hammer test, slake durability test, triaxial test, and direct shear test. While ASTM standards are widely used for testing soils, aggregates, rocks, and other materials, there are also ISRM (International Society of Rock Mechanics) standards that are equally popular worldwide.

Uniaxial Compression Test

Uniaxial compression tests (ASTM D7012) are carried out on rock cores of at least 54 mm diameter (NX cores) with length/diameter ratios of 2–3. The circular ends are flattened and polished so that the load is applied precisely along the axis without any eccentricity, and the longitudinal stress is a principal stress. The specimen is seated on its flat end, and the vertical load is applied and increased gradually, while the longitudinal deformation is measured. From the stress-strain plot thus generated, the uniaxial compressive strength σ_c, Young's modulus E, and the failure strain ε_f are determined.

EXAMPLE 4.3

A diorite specimen obtained from a depth of 360 m below the ground was subjected to a UCS test. The length and diameter of the specimen were 176.6 mm and 63.4 mm, respectively. The mass of the specimen was 1498 g. The stress-strain plot is shown in Figure 4.8. Estimate the uniaxial compressive strength, Young's modulus, failure strain, and the density of the intact rock.

Solution:

It can be seen in Figure 4.8 that no axial load is recorded in the early stages of compression. This occurs due to the closing of microfractures within the rock, some of which are caused by the drilling operation. This trend is common in UCS carried out in rocks where the origin has to be shifted. After the peak stress of 141.5 MPa, which is the UCS, the specimen undergoes brittle failure, with very few readings recorded.

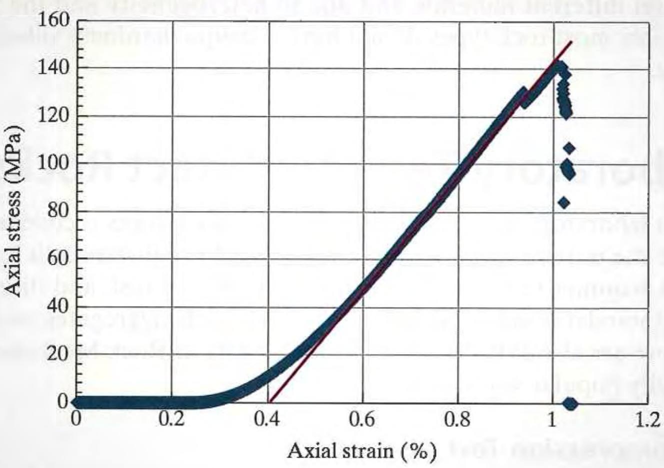
Uniaxial compressive strength, $\sigma_c = 141.5$ MPa

Considering the linear portion of the plot, between axial strains of 0.6 percent and 0.8 percent,

$$E = \frac{\Delta\sigma}{\Delta\varepsilon} = \frac{94 - 47}{0.008 - 0.006} = 23{,}5000 \text{ MPa} = 23.5 \text{ GPa}$$

From the figure, it can be seen that the failure occurs at axial strain of 1.0 percent.

Figure 4.8 Stress-strain plot for the rock specimen in Example 4.3



Indirect Tensile Test

Carrying out a direct tensile strength test on a cylindrical rock specimen is very difficult. First, it is difficult to hold the specimen at the two ends without slippage while pulling. Second, any eccentricity in the applied load can induce an additional moment, which can make the test results questionable. As a result, the *indirect tensile test* (ASTM D3967), also known as the *Brazilian test*, is a preferred alternative for estimating the tensile strength σ_t. Here, a cylindrical specimen in the form of a disc, with length (t)/diameter (d) ratio of about 0.5, is compressed diametrically along the length in the vertical direction, as shown in Figure 4.9. The load is increased gradually to failure. At failure load P, the specimen splits along the vertical diameter into two halves, as if it has been pulled apart horizontally by a tensile force.

From the theory of elasticity and assuming that the rock is isotropic, the tensile strength σ_t is given by (Timoshenko 1934; Hondros 1959)

$$\sigma_t = \frac{2P}{\pi d t} \tag{4.5}$$

The ratio of σ_t from indirect tensile strength test to σ_c typically lies in the range of 1/5 to 1/20, with 1/10 being a good first estimate (Sivakugan et al. 2014). Brazilian indirect tensile strength tests are carried out to determine the indirect tensile strength of concrete cores, but on longer specimens with length/diameter ratios of about 2.

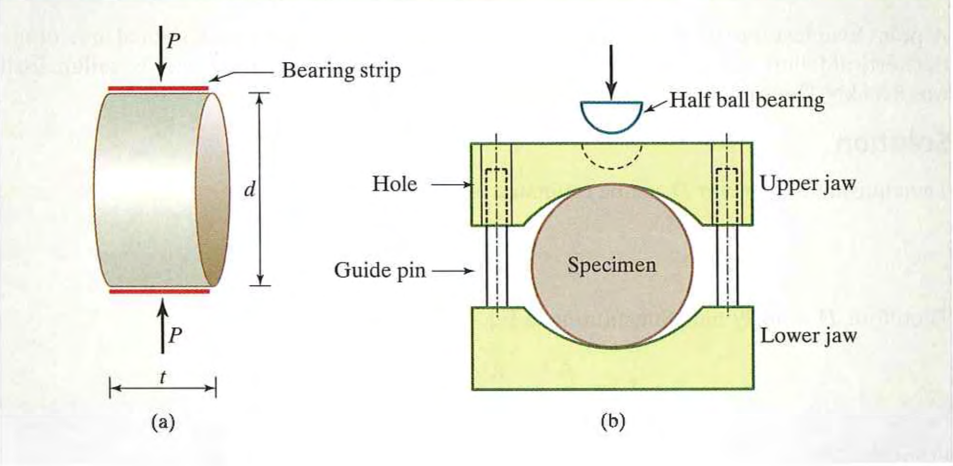
From the values of the σ_t (from indirect tensile strength test) and σ_c, it is possible to determine the shear strength parameters c and ϕ as (Sivakugan et al. 2014):

$$c = \frac{0.5\sigma_c\sigma_t}{\sqrt{\sigma_t(\sigma_c - 3\sigma_t)}} \tag{4.6}$$

$$\phi = \sin^{-1}\left(\frac{\sigma_c - 4\sigma_t}{\sigma_c - 2\sigma_t}\right) \tag{4.7}$$

Indirect tensile strength test

Figure 4.9



(a) (b)

Point Load Test

The *point load test* (ASTM D5731) is a simple and quick test for assessing strength of a rock specimen. The portable testing device consists of two conical loading platens, between which the specimen is held and compressed to failure. The portability of the test equipment and the ability to test irregular specimens that need not be prepared enable the test be carried out at the site on many specimens within a relatively short time. When tested along the diameter (D) of a cylindrical core, the conical spacing is the same as D, and the point load strength index (I_s) is defined as

$$I_s = \frac{P}{D^2} \tag{4.8}$$

For noncircular cross sections, an *equivalent diameter* (see Example 4.4) is used in Eq. 4.8. It has been observed that I_s depends on D, and hence a *size corrected point load strength index* $I_{s(50)}$ is defined as follows:

$$I_{s(50)} = I_s \times \left(\frac{D}{50}\right)^{0.45} \tag{4.9}$$

Here, D is in mm. The size correction standardizes the point load strength index to 50 mm (2.0 in.) diameter specimens. $I_{s(50)}$ values can vary in a range of less than 0.03 MPa (0.3 ton/ft^2) to more than 10 MPa (104 ton/ft^2). σ_c and $I_{s(50)}$ are related by (Bieniawski 1975)

$$\sigma_c = 24\ I_{s(50)}$$

$I_{s(50)}$ is often used as a parameter for classifying rocks based on strength. By carrying out the point load test by loading diametrically and also by loading longitudinally on cylindrical rock specimens, the anisotropy of the rock can be quantified. Anisotropy is the directional variation in the strength and other properties. Typically, σ_t determined from indirect tensile strength test is about 1.25 times $I_{s(50)}$.

EXAMPLE 4.4

A point load test is carried out on an irregular rock specimen. The cross-sectional area of the flat vertical failure surface was measured using graph paper as 1683 mm^2, and the failure load was 8.34 kN. Determine the size-corrected point load strength index $I_{s(50)}$.

Solution

The equivalent diameter D can be estimated as

$$\frac{\pi}{4} \times D^2 = 1683 \text{ mm}^2$$

Therefore, $D = 46.29$ mm. Substituting in Eq. 4.8,

$$I_s = \frac{P}{D^2} = \frac{8340}{46.29^2} = 3.89 \text{ MPa}$$

From Eq. 4.9,

$$I_{s(50)} = 3.89 \times \left(\frac{46.29}{50}\right)^{0.45} = 3.76 \text{ MPa}$$

Schmidt Hammer Test

The *Schmidt hammer* (Schmidt 1951) *test* (ASTM D5873) was originally developed for assessing the hardness of concrete and nowadays is widely used on rocks. It is a small hand-held device that measures the rebound hardness, which can be used in the field as well as in the laboratory. The major advantages are that which is a simple, quick, and nondestructive test that literally costs nothing. The rebound hardness measured by the hammer is a dimensionless number, denoted by *R, H, N,* or *SRH* in the literature. The rebound hardness has been correlated with rock strength parameters.

Slake Durability Test

Even some very hard rocks can deteriorate under repeated wetting and drying cycles in their design life. Their durability is generally assessed through a *slake durability test* (ASTM D4644). Ten rock lumps are placed within a wire basket made of 2 mm mesh, which is partially submerged in a water bath and rotated for ten minutes (Figure 4.10). This is repeated for two or more cycles and the weight loss in the rock sample is a measure of deterioration and hence durability. Generally, the weight loss during the second cycle is taken as the *slake durability index.*

Triaxial and Direct Shear Tests

Triaxial (ASTM D7012) and *direct shear tests* on rocks are similar to those carried out on soils. These tests are required for determining the shear strength parameters cohesion (c) and friction angle (ϕ). Here, the rock specimen is subjected to different

Slake durability apparatus

Figure 4.10

(Courtesy of N. Sivakugan, James Cook University, Australia)

confining pressures and loaded to failure. The shear and normal stresses at failure are used to establish the failure envelope, from which c and ϕ are determined.

4.7 Field Tests for Rocks

Field or in situ tests on rocks generally test a larger volume of the rock mass, which includes the intact rocks and the discontinuities.

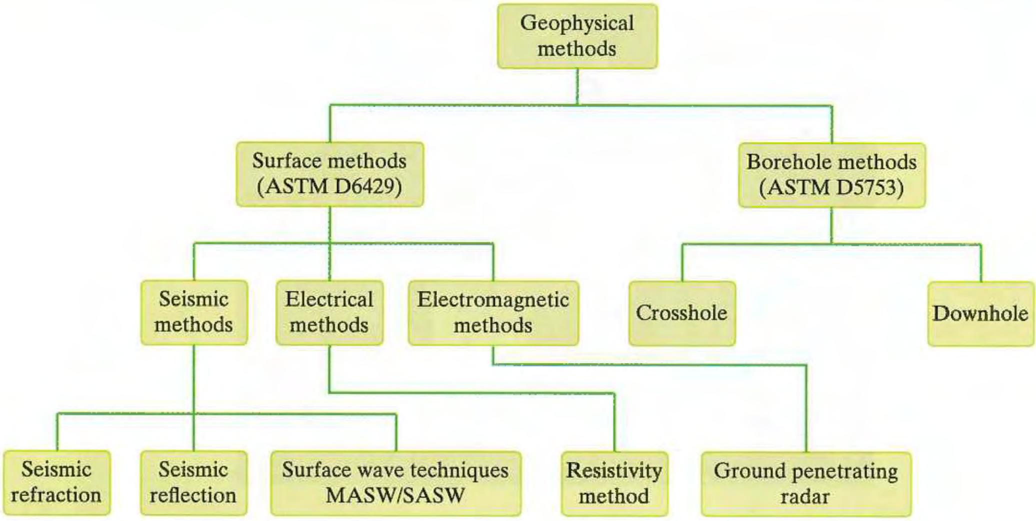
In Situ Tests

One of the main purposes of in situ testing is to measure the current in situ stresses within the rock. Unlike in soils, estimating the lateral in situ stresses within the in situ rock is very difficult, and they are often greater than the vertical normal stresses. They can vary within a wide range. In situ rock stress measurements are required in the designs of rock support systems, predicting rock bursts, stability analysis of underground openings (such as tunnels, caverns, shafts), and determination of excavation methods. Some of the methods commonly used for in situ stress and stiffness measurement are hydraulic fracture (ASTM D4645), overcoring (ASTM D4623), borehole slotting, and flat jacking (ASTM D4729). There are other specialized devices such as dilatometer, pressuremeter, and in situ shear test devices for rocks.

Geophysical Methods

Geophysical methods of soil and rock exploration can effectively complement the data obtained from traditional site investigation through boreholes and trial pits.

Figure 4.11 Geophysical methods for site exploration

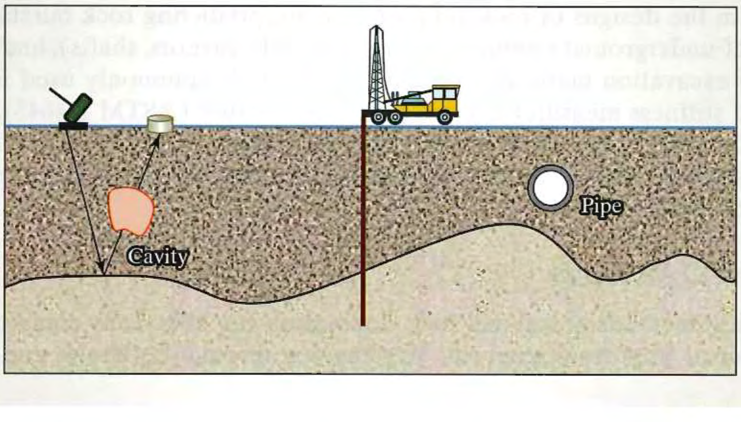


Some of the commonly used geophysical methods are summarized in Figure 4.11. The surface methods (ASTM D6429) are carried out at the ground level, and the borehole methods (ASTM D5753) are carried out within boreholes.

Due to budgetary constraints, the number of boreholes is often kept to the minimum in a site exploration program. When the boreholes are far apart, they can miss some important features such as cavities and buried service lines as shown in Figure 4.12. By geophysical methods like the seismic reflection shown in Figure 4.12, it is possible to detect these and cover larger volumes of soils in a relatively short time.

Seismic methods rely on measuring wave velocities through the rock and soil. A sledgehammer or explosive charge can be used to create the seismic energy source,

Figure 4.12 Geophysical versus geotechnical site exploration



and the arrival times of the waves at different locations are detected by a series of interconnected geophones, from which the wave velocities are computed.

The electrical resistivity method (ASTM D6431) involves inserting two electrodes into the ground and passing current through them. Two more electrodes are placed between them, and the potential difference is measured. From the magnitude of the current, voltage (potential difference), and the electrode spacing, the overall *resistivity (p)* of the soil or rock is measured in $\Omega\cdot$m or ohm·m. Resistivity is a measure of soil or rock resistance to conduct electric current. It is the reciprocal of *conductivity*. Some typical resistivity values of soils and rocks are summarized in Table 4.8. Metals have very low resistivity in the order of $10^{-8}\,\Omega\cdot$m, and insulators have very high resistivity in the order of $10^{16}\ \Omega\cdot$m, with soils and rock lying between the two.

Ground-penetrating radar (ASTM D6432) is one of the common geophysical exploration devices used by engineers, archaeologists, and others for detecting the soil profile, cavities, buried artifacts, service lines, and underground storage tanks. It is a nonintrusive electromagnetic method that works on the basis of the *dielectric constant* of the material through which the electromagnetic pulse goes through. It is a relatively fast method and presents results as the survey progresses.

Seismic downhole (ASTM D7400) and crosshole (ASTM D4428) methods are the two geophysical borehole methods used commonly for soil exploration. Similar to seismic refraction/reflection, the method requires a seismic energy source and one or more geophone receivers. In a downhole test, the seismic source (for example, a sledgehammer hitting a plank) is on the ground surface, and an array of geophone receivers, at 1–3 m intervals, is lowered down a borehole. From the wave velocities measured, the soil profile and the dynamic properties can be determined. In an uphole test, the seismic source is placed in the borehole, and the geophone receivers

Typical electrical resistivity values

Table 4.8

Material	Resistivity $\Omega\cdot$m
Groundwater	10–100 (33–330)
Clay	1–100 (3–330)
Loam	5–200 (15–655)
Clay, sand, gravel mix	40–250 (130–820)
Sands and gravels	50–1000 (165–3,280)
Moist to wet sand	20–200 (65–655)
Weathered rocks	100–1000 (330–3,280)
Shale, limestone, and sandstone	10–1000 (33–3,280)
Crystalline rocks	200–10,000 (655–32,810)
Granite, basalt	50–10,000 (164–32,810)
Marble, quartzite, slate	100–1,000,000 (328–3.28×10^6)
Metamorphic rocks	50–1,000,000 (165–3.28×10^6)
Igneous rocks	100–1,000,000 (330–3.28×10^6)

Note: Values in parentheses are in $\Omega\cdot$ft.

are placed at the ground level. Uphole or downhole tests require only one borehole and are hence more economical than the crosshole ones, which require two or more boreholes.

4.8 Rock Mass Classification

The discontinuities have a major influence on the behavior of the rock mass. Very often, their influence is more than that of the intact rock. What we test in the laboratory is the intact rock, which says very little about the discontinuities. The characteristics of the discontinuities are determined at the site. The characteristics of discontinuities that affect the rock mass behavior are summarized in Table 4.9.

With so many different parameters reflecting the characteristics of discontinuities in addition to the strength of the intact rock, there is certainly a need to have a system for classifying the rock mass. The classification system ensures we all speak the same language in a rock mechanics sense. Three of the most used rock mass classifications are:

a. Rock mass rating (RMR) proposed by Bieniawski (1973)
b. Q-system proposed by Barton et al. (1974)
c. Geological strength index (GSI) proposed by Hoek (1994)

In these classification systems, based on the characteristics of the discontinuities and the intact rock, a numerical value is assigned to reflect the behavior of the rock mass.

4.9 Rockfills

Rockfills consist of gravel-sized aggregates to large boulders. They are used generally in underground mine backfilling and in the construction of dams, spillways, and upstream slope protection. Most of the large rockfill dams worldwide are relatively

Table 4.9 Rock discontinuity parameters

Parameter	Remarks
Orientation	The orientation of the discontinuity with respect to the proposed work can affect the stability significantly.
Spacing, No. of joint sets, Block size	Spacing of the joints, number of joint sets, and block size reflect the degree of fragmentation, and the degrees of freedom for the rock mass to deform.
Aperture Filling	The width of the discontinuities, whether they are closed, open, or filled, and the characteristics of the filling material play a role on the stability.
Wall strength, Roughness	The profile of the interface and the roughness must be considered.

Laboratory tests on rockfills: (a) triaxial test, (b) direct shear test

Figure 4.13

(a) (b)

(Courtesy of N. Sivakugan, James Cook University, Australia)

recent. The laboratory and field tests for rockfills are very similar to those for soils and aggregates, the main difference being the scale. The equipment used for rockfill tests in the laboratory are orders of magnitude larger so that the tests are more realistic. Figure 4.13a shows a 300 mm diameter representative rockfill specimen being prepared for a triaxial test. Figure 4.13b shows a large direct shear box test on a 1000 mm × 1000 mm × 600 mm specimen.

4.10 Summary

1. Rocks are relevant to civil engineers in two ways: (a) as a construction material and (b) the stability of slopes, tunnels, and foundations involving rock mechanics.
2. The most common building stones are basalt, granite, sandstone, limestone, shale, marble, and slate.
3. Seismic waves produced during blasting or earthquakes can be of two types, namely, body waves (e.g., P-waves and S-waves) and surface waves (e.g., Rayleigh waves and Love waves).
4. Discontinuity in a rock mass includes joint, fault, bedding plane, cleavage, and schistosity. It breaks the continuity of the rock mass, causing the rock mass to be weaker and more permeable than the intact rock.
5. RQD is an index in the range of 0–100, which is a measure of how fragmented the rock mass is.

6. Uniaxial compressive strength σ_c can vary in the range of 50–350 MPa, in comparison with 30–40 MPa for concrete.
7. A P-wave is also called a longitudinal wave, pressure wave, or compression wave. An S-wave is also called secondary wave or shear wave. P-waves can travel through solids, liquids, and gases; S-waves can travel only through solids.
8. Some of the important design parameters for intact rock are $\sigma_c, \sigma_t, E, \nu, I_{s(50)}, c$, and ϕ.
9. The greater the resistivity, the greater the resistance to conducting electric current.
10. Rockfills are used in construction of dams and for backfilling underground mine voids.

Exercises

1. Which of the following is not commonly used as a dimension stone?
   - **a.** Granite
   - **b.** Gypsum
   - **c.** Sandstone
   - **d.** Marble
2. Which of these waves is the fastest?
   - **a.** Compression wave
   - **b.** Shear wave
   - **c.** Rayleigh wave
   - **d.** Love wave
3. Which of the following rock types is least preferred for sculpture?
   - **a.** Sandstone
   - **b.** Marble
   - **c.** Slate
   - **d.** Basalt
4. What is the most common rock core size preferred for laboratory testing?
   - **a.** EX (22.2 mm diameter)
   - **b.** AX (30.2 mm diameter)
   - **c.** BX (41.3 mm diameter)
   - **d.** NX (54.0 mm diameter)
5. Which of the following rock types would generally have the greatest uniaxial compressive strength?
   - **a.** Basalt
   - **b.** Sandstone
   - **c.** Limestone
   - **d.** Shale
6. A marble specimen has uniaxial compressive strength of 70 MPa. A likely value for its Young's modulus is
   - **a.** 500 MPa
   - **b.** 5 GPa
   - **c.** 50 GPa
   - **d.** 500 GPa

7. Typical value for P-wave velocity in rocks is

a. 4 m/s
b. 40 m/s
c. 400 m/s
d. 4000 m/s

8. A rock specimen has uniaxial compressive strength of 150 MPa. A likely value for its indirect tensile strength from the Brazilian indirect tensile test would be

a. 15 kPa
b. 15 MPa
c. 150 MPa
d. 150 GPa

9. Which of the following is not a geophysical method for site exploration?

a. Standard penetration test
b. Electrical resistivity method
c. Seismic refraction test
d. Seismic crosshole method

10. Which of the following has the lowest electrical resistivity?

a. Metal
b. Soil
c. Rock
d. Rubber

11. State whether the following are true or false.

a. The Taj Mahal is made of granite.
b. Stonehenge is made of marble.
c. A rock mass is always weaker than the intact rock.
d. A rock mass is more permeable than the intact rock.
e. Compression waves (P-waves) are always faster than shear waves (S-waves).
f. P and S waves are both body waves.
g. P-waves cannot travel through water.
h. The denser the rock, greater the wave velocity is.
i. For UCS and indirect tensile strength tests, the core should be at least of NX size.
j. Rocks have greater resistance to conducting current than soils.

12. Name two common rock types for igneous, sedimentary, and metamorphic rocks that are used in construction. Which two of these are used for sculpture?

13. What is the difference between quick lime and slaked lime?

14. What is a sinkhole? How are they formed?

15. Surf the web and look for images of discontinuities. In your own words, write a 200-word essay on discontinuities in rocks, emphasizing the differences between faults, joints, bedding planes, schistosity, and cleavages.

16. A granite rock specimen in the form of a rectangular specimen, with dimensions of 250 mm (height) × 100 mm × 100 mm, is subjected to principal stresses of 30 MPa, 15 MPa, and 15 MPa in the vertical and horizontal directions, respectively. Assuming that these three stresses are principal stresses (along x-, y- and z- directions), estimate the new dimensions of the specimens under the applied stresses. (Hint: Use Eq. 1.15, and assume values for E and ν.)Determine the volumetric strain (ε_{vol}) from Eq. 1.22, and check whether it is the same as $\varepsilon_x + \varepsilon_y + \varepsilon_z$.

17. Using Eqs. 1.14, 1.19, 4.3, and 4.4, show that

$$\frac{v_p}{v_s} = \sqrt{\frac{1-\nu}{0.5-\nu}}$$

18. What is the difference between rock mass and intact rock?

19. Carry out a literature review and compare the two surface wave techniques for site exploration MASW and SASW.

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5 CHAPTER

Concrete, used in the construction site shown, contains both fine and coarse aggregates, mixed with water and cement. Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology

Aggregates

5.1 Introduction

The term *aggregate* generally refers to coarse particulate rock-like material consisting of a collection of disparate elements of size ranging from a small fraction of a millimeter (e.g., smallest particle size less than 0.1 mm) to tens of millimeters (e.g., upto 50 mm). It includes natural gravel, crushed rock, sand, recycled concrete, slag, and synthetic or artificial (human made) aggregate. Depending on the particular application, an aggregate used in civil engineering construction may contain a small amount of fine-grained materials, such as clay and silt. Typical examples of aggregates used in civil engineering construction are shown in Figure 5.1.

Aggregates are key constituents of cement concrete and asphalt concrete (AC) or hot mix asphalt (HMA) that are used in the construction of a wide range of civil engineering structures such as buildings and roads. Typically, aggregates constitute 60–75 percent by volume and 79–85 percent by mass in portland cement concrete (PCC). Similarly, 75–85 percent by volume and 92–96 percent by mass is made of aggregates in AC. Moreover, aggregates are used as the base or subbase of a road pavement, ballast for railroads, and permeable material in a drainage system (such as surrounding perforated pipes in trench drains and as the drainage layer behind retaining walls to transmit water). When mixed with appreciable quantity of portland cement or asphalt cement or other cementitious material (like slag, lime, and fly ash) to provide significant cementing, or binding, the aggregate is commonly referred as *bound material*. On the other hand, when used without any significant amount of asphalt cement or portland cement or other cementitious binding material, it is referred as *unbound material*.

Aggregates are often divided into two groups: *coarse-grained* and *fine-grained*. Aggregate with particles retained on a sieve with opening size 4.75 mm (using a No. 4 sieve) is referred as *coarse-grained aggregate*, and that which passes through 4.75 mm size sieve is called *fine-grained aggregate*.

Typical examples of aggregates: (a) a road base material (b) 10 mm crushed rock aggregate used for making concrete

Figure 5.1

(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Sources of natural aggregates include gravel pits, river deposits, and rock quarries. Certain manufactured or synthetic aggregates, either lightweight or heavyweight, are also used in civil engineering construction. In addition, recycled or reclaimed aggregates, such as crushed PCC and clay bricks, are also used in significant quantities. Nevertheless, crushed rock from quarries is by far the most common source for aggregates used in civil engineering construction.

5.2 Origin, Geology and Classification of Parent Rocks

Aggregates are commonly obtained by crushing naturally occurring rock in a quarry such as that shown in Figure 5.2. These are referred to as crushed rock aggregates, and they are the most common source of aggregates used in civil engineering construction. Their properties are dependent on the parent rock. The parent or source rocks are classified in accordance with their geologic origin as igneous, metamorphic, or sedimentary.

Igneous rocks are produced by cooling and crystallization of hot molten magma formed by volcanic action. Magma consists chiefly of molten silicates, and it is extremely viscous and of high density. The Si:O ratio in the magma determines which minerals will crystallize. As the magma cools, either at the earth's surface or at depth, it crystallizes in a specific order, known as Bowen's reaction series. Igneous rocks are often classified on the basis of crystal grain size; grain size more than 2 mm is referred as coarse, grain size less than 0.2 mm as fine, and the sizes in between as intermediate. Cooling at the surface occurs rapidly and produces extrusive igneous rocks, typically with fine grains (such as basalt, andesite, and rhyolite) and cooling at depth is relatively slow and produces intrusive igneous rocks with coarse grains (such as granite, diorite, and gabbro).

Figure 5.2 Photo of a typical quarry from which aggregate is sourced

(Courtesy of N. Sivakugan, James Cook University, Australia)

Sedimentary rocks are derived from the accumulation of weathered rock products and fossils deposited in water and air on the earth's surface and the subsequent hardening and/or cementation into rock form. They are generally formed in horizontal, parallel beds referred as strata. There are three different types: namely clastic, chemical, and organic sedimentary rocks. Clastic sedimentary rocks are formed by the accumulation of particles physically transported to a depositional environment, compacted through burial, and cemented by mineral precipitation (examples: conglomerate, breccia, sandstone, siltstone, shale). Chemical sedimentary rocks are formed by precipitation of salts from marine waters (examples: $CaCO_3$—limestone (calcite), SiO_2—chert (quartz), NaCl—halite, $CaSO_4$—gypsum). Organic sedimentary rocks are formed through the accumulation of shells, coral, plant matter, plankton, and so forth (examples: coquina, chalk, coal).

Metamorphic rocks result from metamorphism, which refers to the process whereby existing (precursor) rocks get altered to new forms due to high temperature and/or pressure. The precursor rock may be sedimentary, igneous, or metamorphic rock, and the degree of metamorphism controls the mineralogical composition and texture of the resultant rock (examples: shale → slate → phylite → schist → gneiss → migmatite; basalt → greenstone → greenschist → amhibolite → pyroxene granulite; sandstone → quartzite; limestone → marble → crystalline limestone).

Aggregates from all three classes of rocks are used in civil engineering construction for different applications. However, suitability of aggregates from a source for a particular application is evaluated through various tests on samples of the rock for physical, mechanical, and chemical characteristics.

Mineralogy, grain size and texture, and petrographic description of rock samples are also used in the evaluation of the source rock. Due to the nature of formation, the source rock could be largely variable in properties, and tests are usually done on small samples. Hence, the evaluation process involves testing multiple samples, covering the full extent of the source rock, and the decision concerning suitability is based on historical evidence of the performance of aggregates from different source rocks in similar applications.

5.3 Properties and Testing of Aggregates

Properties of an aggregate are expressed by the characteristics of the individual particles as well as the characteristics of the collective material. Physical, chemical, and mechanical properties of an aggregate are assessed and checked against the requirements for a particular application or project before being accepted for use. Depending on the use of the aggregate, different properties could be important, and the relative importance of various properties of aggregate for typical uses is summarized in Table 5.1. Some of the important properties of an aggregate for common use

Importance of basic aggregate properties for different uses

Table 5.1

	Relative importance for use in*		
Property	PCC	AC	Road base material
PHYSICAL PROPERTY			
Particle size—distribution, maximum	M	M	M
Particle shape (flakiness, elongation)	M	M	M
Particle shape (angularity)	M	V	V
Particle surface texture	M	V	V
Pore structure, porosity	V	M	U
Specific gravity, absorption	V	M	M
Soundness—withstand weathering	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 PROPERTY			
Solubility	M	U	U
Surface charge	U	V	U
Asphalt cement affinity	U	V	M
Reactivity to chemicals	V	U	U
Volume stability—chemical	V	M	M
Coatings	M	M	U
MECHANICAL PROPERTY			
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*

(Based on Mamlouk, M.S and Zaniewski, J.P. (2011). Materials for civil and construction engineers. Third Edition. Pearson Education Inc., New Jersey, USA.)

in civil engineering projects are discussed further in this section. It is worth noting that several standard tests have been developed by organizations such as ASTM for determining the aggregate properties, and they are identified in this discussion so that the reader could refer to the relevant standards for further details.

5.3.1 Particle Size and Grading

Gradation or particle size distribution indicates the weight proportion of different size particles in the aggregate. This is commonly determined by sorting the different size fractions of the aggregate sample by sieving through a stack of sieves arranged in descending order of their opening size from top to bottom and weighing the amount retained in each sieve. Typical examples of different opening size sieves and vibrating equipment (commonly referred as a "sieve shaker") used for sieve analysis of a small sample are shown in Figure 5.3.

Nominal opening size of standard sieves for analyzing the particle size distribution of coarse aggregates, mm (in.): 125 (5), 100 (4), 90 $\left(3\frac{1}{2}\right)$, 75 (3), 63$\left(2\frac{1}{2}\right)$, 50 (2), 37.5 $\left(1\frac{1}{2}\right)$, 25.0 (1), 19.0 $\left(\frac{3}{4}\right)$, 12.5 $\left(\frac{1}{2}\right)$, 9.5 $\left(\frac{3}{8}\right)$. Smaller opening–size sieves used for analyzing the particle size distribution of fine aggregate are generally referred by numbers. The U.S. Standard sieve numbers and the corresponding opening sizes for commonly used sieves are listed in Table 5.2.

Figure 5.3 Typical sieves and vibrating equipment used for sieve analysis: (a) U.S. No. 4 (meaning that the opening size is 4.75 mm) sieve; (b) U.S. No. 50; (opening size 300 μm) sieve; (c) sieve stack under vibration in a sieve shaker.

(a) (b) (c)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Table 5.2 U.S. Standard Sieve No. and corresponding opening size of sieve for fine aggregate:

U.S. Standard Sieve No.	Sieve opening size (mm)
4	4.75
5	4.00
6	3.35
7	2.80

8	2.36
10	2.00
12	1.70
14	1.40
16	1.18
18	1.00
20	0.850
30	0.600
40	0.425
50	0.300
60	0.250
70	0.212
80	0.180
100	0.150
140	0.106
200	0.075
230	0.063
270	0.053
325	0.045
400	0.038

The weight percent of material passing each sieve is calculated from the measurement of retained material on each sieve. The variation of percent passing plotted against the sieve size (that is, particle size, commonly referred as particle diameter) in log-scale is the gradation curve of the aggregate. A typical gradation plot of an aggregate sample is shown in Figure 5.4.

It is worth noting that not all the sieves listed above are generally needed for determining the gradation of a particular aggregate. Selection of the (opening sizes of) sieves required for analyzing the gradation of a particular sample of aggregate depends on the maximum size and gradation of that material. Moreover, fine particles such as silt and clay could adhere to larger size particles, leading to inaccurate gradation results, and wet sieving may be appropriate for testing the gradation of such unclean materials. Further details regarding sieve analysis of aggregate could be found in ASTM C136 and similar standards.

EXAMPLE 5.1

A sample of fine aggregate was tested for its gradation, and the results are given below. Calculate the percent passing for each sieve and plot the particle size distribution of the aggregate.

The following size sieves were used, and the net weights of the material retained on each of them are:

Sieve opening size (mm)	4.75	2.36	1.18	0.6	0.3	0.063	0.02	0.006	Pan
Amount retained (g)	0	32.5	57.0	61.8	63.9	157.7	95.9	53.3	10.6

Table 5.3 Calculation of percent passing for each sieve size

Sieve opening size (mm)	Retained amount (g)(A)	Cumulative retained amount (g)(B)	Cumulative retained amount (percent) ($C = \frac{B}{Total} \times 100$)	Percent finer or Percent passing ($D = 100 - C$)
4.75	0	0	0	100
2.36	32.5	32.5	6.1	93.9
1.18	57.0	89.5	16.8	83.2
0.6	61.8	151.3	28.4	71.6
0.3	63.9	215.2	40.4	59.6
0.063	157.7	372.9	70.0	30.0
0.02	95.9	468.8	88.0	12.0
0.006	53.3	522.1	98.0	2.0
Pan	10.6	532.7	100	
Total amount	532.7			

Solution

Step 1: Record the given data in table form in the left two columns as shown in Table 5.3.

Step 2: Calculate the cumulative retained amount (or the amount larger than a particular size), either by using spreadsheet software or by hand.

For example: amount retained on 4.75 mm sieve + amount retained on 2.36 mm sieve = $0 + 32.5$ g $= 32.5$ g, which is recorded as the cumulative amount corresponding to the 2.36 mm sieve in column B; the amount larger than 1.18 mm = the amount larger than 2.36 mm + amount retained on 1.18 mm sieve $= 32.5 + 57.0$ g $= 89.5$ g.

Step 3: The percent of cumulative retained amount (C) for each sieve opening size could then be calculated by dividing the cumulative retained amount by the total amount and multiplying by 100 ($C = \frac{B}{Total\ amount} \times 100$).

Step 4: The "percent finer than" or "percent passing" a particular (sieve) size (D) is then calculated as 100 minus the cumulative percent retained (that is, the percent larger than the particular size) calculated in C, so $D = 100 - C$.

The following example illustrates the determination of the gradation of aggregate and the use of the results.

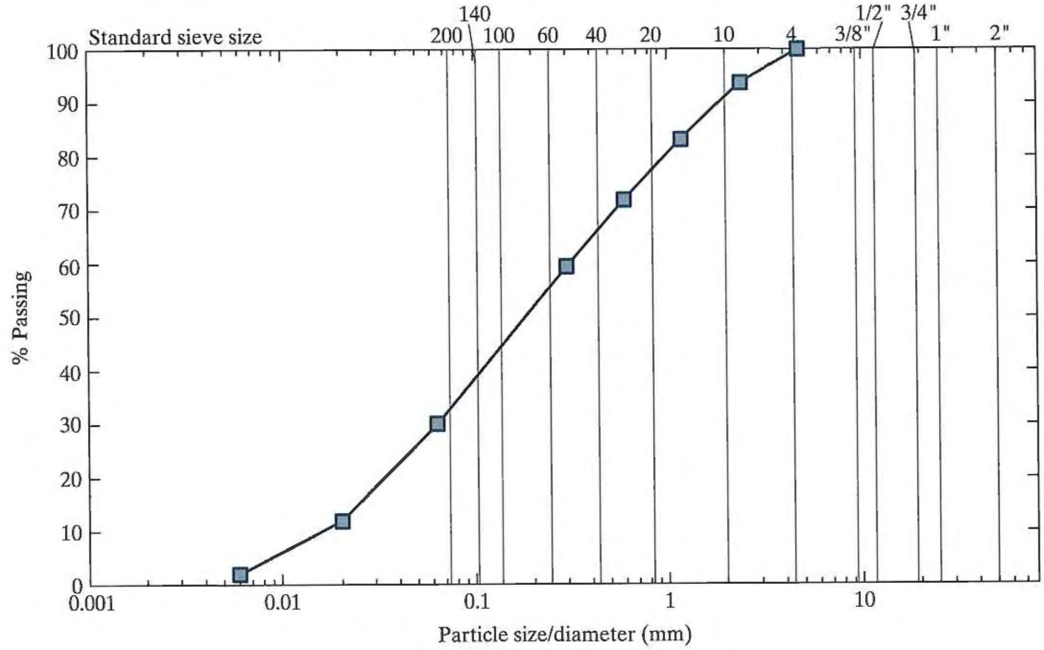
The grading curve of the aggregate is then obtained by plotting the percent passing (column D) against the sieve opening size (also referred as particle size or particle diameter) in the first (left) column in a semi-log graph as shown in Figure 5.4.

5.3.2 Types of Grading and Relationship with Density

Gradation is an important characteristic of aggregate that influences how it performs in a civil engineering application such as the unbound granular base of a road. When compacted aggregate is used as a road base material, how dense it will pack into and the strength, stiffness, and permeability of the compacted aggregate road base

Typical gradation curve of aggregate plotted in semi-log graph

Figure 5.4



are highly influenced by its gradation. In general, the denser the aggregate material, the higher the strength and stiffness is, and the lower the permeability (due to the reduced void space for transmitting water).

In 1907, Fuller and Thompson developed an equation for the maximum density gradation of the aggregate:

$$P_i = \left(\frac{d_i}{D_{\max}}\right)^n \times 100 \tag{5.1}$$

Where

P_i = percent passing corresponding to particle size d_i

$D_{\max}$ = maximum particle size

d_i = particle size, and

n is a parameter that adjusts for the fineness and coarseness of the aggregate.

$n \approx 0.5$, according to Fuller and Thompson

The Federal Highway Administration (FHWA) revised the equation proposed by Fuller and Thompson and recommended that the power of the equation n be 0.45 for maximum density grading.

Definitions of the Common Terms

Maximum size of aggregate: The size of the smallest sieve through which 100 percent aggregate particles passes.

Nominal maximum size of aggregate: Size of the largest sieve that retains less than 10 percent (by weight) of the aggregate particles. *Superpave* defines the nominal maximum size of aggregate as "one sieve size larger than the first sieve to retain more than 10 percent of the material" (Roberts et al. 1996).

The following example illustrates the calculations involved in developing the maximum density grading of aggregate.

EXAMPLE 5.2

Calculate the percentage passing for each common sieve size and plot the maximum density grading curve of aggregate that has a maximum size of 25 mm (1 in.) in accordance with FHWA recommendations.

Solution:

The maximum particle size, $D_{\max} = 25$ mm.

The common sieve sizes and calculations of percent passing each sieve using the 0.45 power law are detailed in Table 5.4. This calculation could easily be performed using a spreadsheet program.

Table 5.4 Calculation of percent passing each sieve size for maximum density gradation

Particle size/diameter or sieve size, d_i (mm)	Percent passing, $P_i = \left(\frac{d_i}{D_{\max}}\right)^n \times 100$
25	$P_{25} = \left(\frac{25}{25}\right)^{0.45} \times 10.0 = 100$
19	$P_{19} = \left(\frac{19}{25}\right)^{0.45} \times 100 = 88.4$
12.5	$P_{12.5} = \left(\frac{12.5}{25}\right)^{0.45} \times 10.0 = 73.2$
9.5	$P_{9.5} = \left(\frac{9.5}{25}\right)^{0.45} \times 10.0 = 64.7$
4.75	$P_{4.75} = \left(\frac{4.75}{25}\right)^{0.45} \times 10.0 = 47.4$
2.36	$P_{2.36} = \left(\frac{2.36}{25}\right)^{0.45} \times 10.0 = 34.6$
1.18	$P_{1.18} = \left(\frac{1.18}{25}\right)^{0.45} \times 10.0 = 25.3$
0.6	$P_{0.6} = \left(\frac{0.6}{25}\right)^{0.45} \times 10.0 = 18.7$

0.3	$P_{0.3} = \left(\frac{0.3}{25}\right)^{0.45} \times 10.0 = 13.7$
0.15	$P_{0.15} = \left(\frac{0.15}{25}\right)^{0.45} \times 10.0 = 10.0$
0.075	$P_{0.075} = \left(\frac{0.075}{25}\right)^{0.45} \times 10.0 = 7.3$

The maximum density gradation curve of aggregate is obtained by plotting percent passing (P_i) against particle size (d_i) as shown in Figure 5.5.

Types of Grading

A few terms concerning the grading of aggregate that are commonly used in civil engineering applications are:

Dense- or *well-graded aggregate*: Aggregate with gradation close to FWHAs 0.45 power maximum density grading curve.

Gap-graded: Aggregate that has only a small percentage of a particles in the intermediate-size (or mid-size) range. The grading curve of such aggregate is almost flat in the mid-size range.

Uniformly graded: Aggregate that contains particles mostly of a size in a very narrow range. The grading curve of such aggregate is steep (almost vertical) over a very narrow size range. All the particles of such aggregate are basically of the same size, and this is also referred as *one-sized* aggregate.

Open-graded: Aggregate that has only small percentage of particles in the small-size range.

These grading types are illustrated in the particle size distribution plot in Figure 5.6.

Grading Specifications

As discussed earlier, grading is an important characteristic of aggregate which could easily be assessed from simple testing and used for quality assurance/control. Therefore, grading specifications have been developed by the relevant organizations, such as AASHTO, ASTM, FWHA, Federal Aviation Administration (FAA), the Asphalt Institute, and so forth, for different civil engineering applications.

Aggregate is commonly used for constructing the base, subbase, and/or surface course of a road, and AASHTO, ASTM, and other organizations (for example, the Department of Transport of various different states) have proposed grading and other requirements of aggregate for such use. For instance, the grading standards of aggregate materials specified in ASTM-D1241 for road base, subbase, and/or surface course construction are presented in Table 5.5. Depending on the location, traffic loading and the class or importance of the road as well as the layer in which the aggregate material is used (such as the base, subbase, or surfacing), a particular specified

Figure 5.5 Maximum density gradation curve of 25 mm maximum size aggregate (a) in semi-log gradation format (b) in 0.45 power-grading chart format

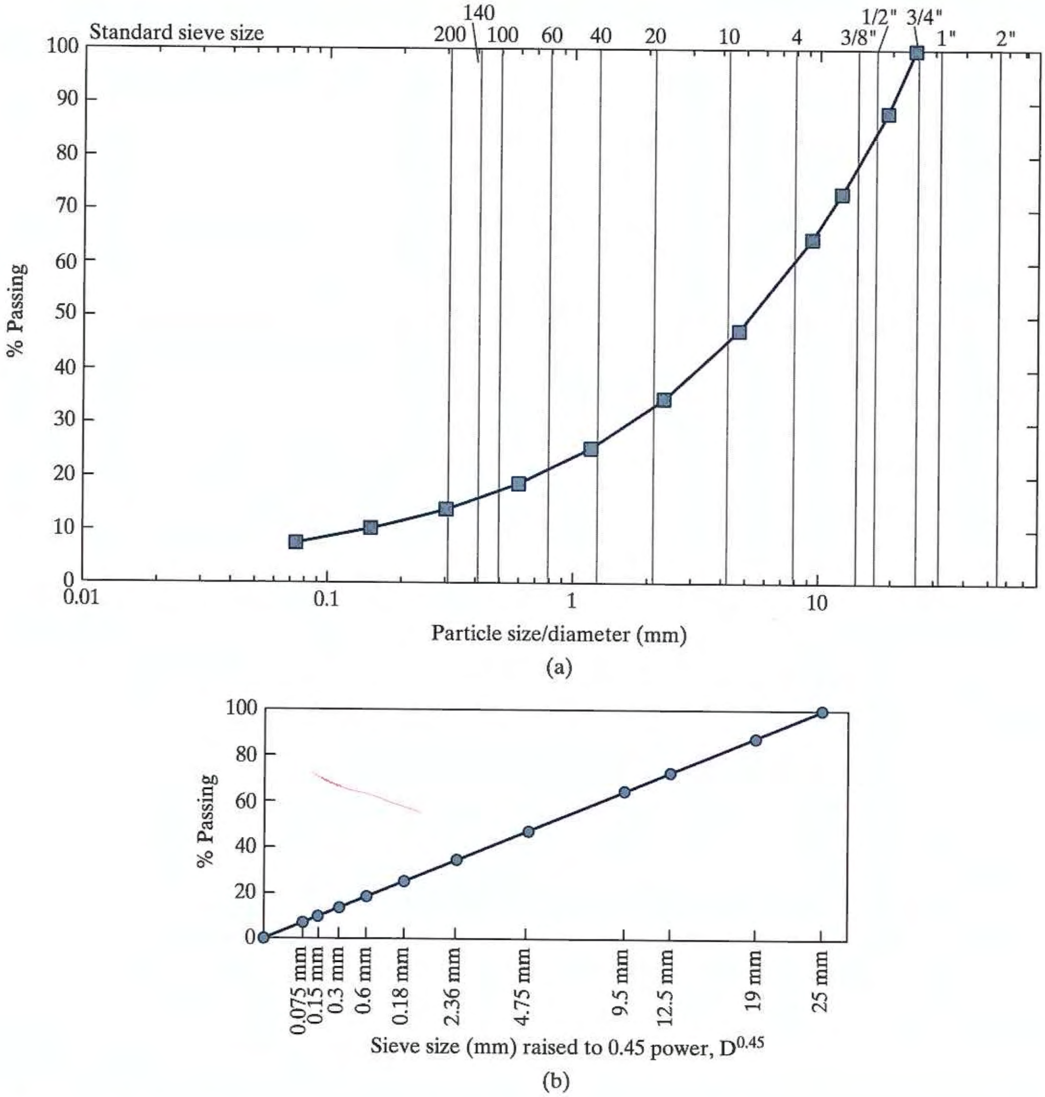


Figure 5.6 Illustration of different gradation types of aggregate

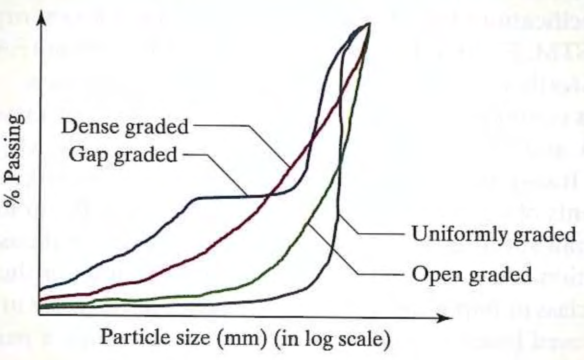


Table 5.5

Gradation requirements for soil-aggregate materials for roads (from ASTM-D1241)

Sieve size (square openings)	Weight percent passing square mesh sieves					
	Type I				Type II	
	Gradation A	Gradation B	Gradation C	Gradation D	Gradation E	Gradation F
50 mm (2-in)	100	100	—	—	—	—
25 mm (1-in)	—	75–95	100	100	100	100
9.5 mm (3/8-in)	30–65	40–75	50–85	60–100	—	—
4.75 mm (No. 4)	25–55	30–60	35–65	50–85	55–100	70–100
2.0 mm (No. 10)	15–40	20–45	25–50	40–70	40–100	55–100
425 μm (No. 40)	8–20	15–30	15–30	25–45	20–50	30–70
75 μm (No. 200)	2–8	5–15	5–15	8–15	6–15	8–15

(Based on ASTM-D1241)

grading given in Table 5.5 is chosen. In addition to grading, there are other requirements that need to be satisfied for a particular use, and further details can be found in ASTM-D1241.

Aggregate is the major constituent of AC, and the gradation requirements of aggregate for Superpave AC mix is given in Table 5.6 (from McGennis et al. 1995 and Harman et al. 2002). Similarly, aggregate is about 79–85 percent by weight of PCC, which consists of coarse aggregate and fine aggregate. Fine aggregate requirements for PCC as specified in ASTM-C33 are presented in Table 5.7. Selected grading requirements of coarse aggregate for PCC as specified in ASTM-C33 are presented in Table 5.8.

Table 5.6

Grading requirement for Superpave asphalt concrete

Sieve size	Percent passing				
	Nominal maximum sieve size				
	9.5 mm	12.5 mm	19 mm	25 mm	37.5 mm
50.0 mm (2 in.)					100
37.5 mm (1.5 in.)				100	90–100
25.0 mm (1 in.)			100	90–100	90 max
19.0 mm (3/4 in.)		100	90–100	90 max	
12.5 mm (1/2 in.)	100	90–100	90 max		
9.5 mm (3/8 in.)	90–100	90 max			
4.75 mm (No. 4)	90 max				
2.36 mm (No. 8)	32–67	28–58	23–49	19–45	15–41
1.18 mm (No. 16)					
75 μm (No. 200)	2.0–10.0	2.0–10.0	2.0–8.0	1.0–7.0	0.0–6.0

(From McGennis, R.B., Anderson, R.M., Kennedy, T.W. and Solaimanian, M. (1995). Background of superpave – Asphalt mixture design and analysis. Technical report: Federal Highway Administration FWHA-SA-95-003, Washington, DC 20590 and Harman, T., D'Angelo, J., Bukowski, J. and Paugh, C. (2002). Superpave Asphalt Mixture Design Workshop Workbook – Ed. Shenoy, A. U.S. Department of Transportation, Federal Highway Administration.)

Table 5.7 Grading requirements of fine aggregate for PCC (ASTM-C33)

Sieve size	Percent Passing
9.5 mm (3/8 in.)	100
4.75 mm (No. 4)	95–100
2.36 mm (No. 8)	80–100
1.18 mm (No. 16)	50–85
600 μm (No. 30)	25–60
300 μm (No. 50)	5–30
150 μm (No. 100)	0–10
75 μm (No. 200)	0–3.0

(Based on ASTM-C33)

Table 5.8 Selected grading requirements of coarse aggregate for PCC from ASTM-C33

Sieve size (Squire openings)	Percent passing by mass			
	Nominal size (aggregate size number)			
	37.5–4.75 (467)	25.0–4.75 (57)	19.0–4.75 (67)	12.5–4.75 (7)
50 mm (2 in)	100			
37.5 mm (1.5 in)	95–100	100		
25 mm (1 in)	—	95–100	100	
19.0 mm (3/4 in)	35–70	—	90–100	100
12.5 mm (1/2 in.)	—	25–60	—	90–100
9.5 mm (3/8 in.)	10–30	—	20–55	40–70
4.75 mm (No. 4)	0–5	0–10	0–10	0–15
2.36 mm (No. 8)		0–5	0–5	0–5

(Based on ASTM-C33)

Mixing Different Aggregates to Obtain the Required Grading

Having seen the aggregate grading requirements for different civil engineering applications, it is worth noting that it is not always possible to satisfy the required grading for a particular need for various practical reasons. For example, a large quantity of a particular aggregate may be available, but it may not meet the specified grading requirements for constructing a road base. Alternatively, there could be two more types of aggregates available, but none of them meet the grading specification. In such cases, blending or mixing two or more aggregates to satisfy the required grading could be a good option.

Blending of aggregates to meet a specified grading requirement is performed by mixing the two or more types of aggregates in accordance to predetermined mass proportion. Determining the mixing proportion of the aggregates could be performed by a graphical method such as that given in RRL (1968). However, with the availability of computers and spreadsheet software, the blending proportion could be determined quite elegantly through an iterative procedure, and it is illustrated through the following example.

EXAMPLE 5.3

A road base is to be constructed using unbound gravel material, and the specified grading for the base is given in the left 3 columns of Table 5.9. Two aggregate materials A and B are available, and the grading of each has been determined from sieve analysis and the results are tabulated in columns (E) and (F) in Table 5.9. Determine the proportion that could be adopted for blending the two aggregates A and B to satisfy the specified grading.

Solution:

If the blending proportion of Material A is *a* percent, then the proportion of Material B is $(100 - a)$ percent. It is noted that *a* could vary from 0–100.

Assume a value for *a* and calculate Column (*G*) using,

$$G_i = \frac{a}{100} \times E_i + \frac{(100 - a)}{100} \times F_i \tag{5.2}$$

Note: Subscript *i* identifies the values in the same row of Table 5.9.

Compare the calculated values in each row of Column G and verify whether they are in between the corresponding lower and upper limits of the specified grading recorded in Columns C and D respectively. If not, change the value of *a* and proceed until the values in Column G are in between those in Columns C and D.

By changing the value of *a* in a systematic fashion, you could arrive at an acceptable solution.

For a value of $a = 50$, the values in Column G are found to be in between the limits in Columns C and D. Therefore, 50–50 blending of Materials A and B is suitable.

However, there could be other possibilities, and 40 percent A + 60 percent B is also an alternate solution (see Column H of Table 5.9).

It is a good practice to plot the grading curves also of the blend and the specified materials and confirm the results as illustrated in Figure 5.7.

Particle size distribution data of specified, available, and blended aggregates — Table 5.9

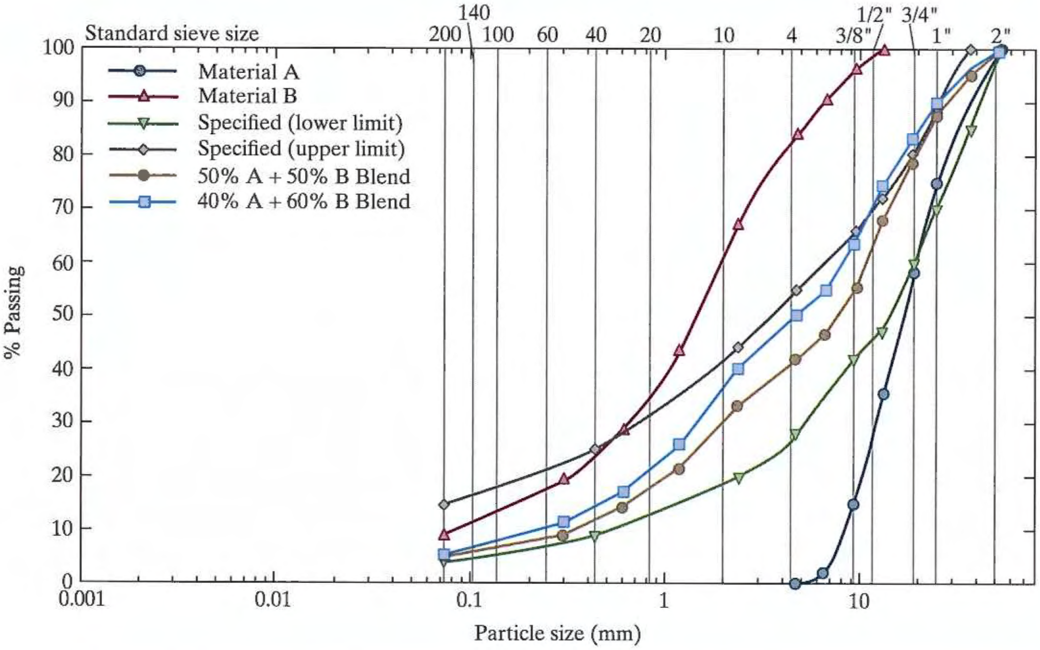
Sieve size (mm)	Percent passing (by mass)					
	Specified grading		Material A	Material B	50% A + 50% B Blend	40% A + 60% B Blend
	Lower limit	Upper limit				
	C	D	E	F	G	H
53	100	100	100	100	100	100
37.5	85	100	90.1	100	95.05	96.04
25	70	90	75.2	100	87.6	90.08
19	60	80	57.6	100	78.8	83.04
13.2	48	72	35.9	100	67.95	74.36
9.5	42	66	14.9	96	55.45	63.56
6.7			2.5	90.6	46.55	55.36

(*Continues*)

Table 5.9 Particle size distribution data of specified, available, and blended aggregates (*Continued*)

Sieve size (mm)	Percent passing (by mass)					
	Specified grading		Material A	Material B	50% A + 50% B Blend	40% A + 60% B Blend
	Lower limit	Upper limit				
	C	D	E	F	G	H
4.75	28	55	0.1	83.9	42	50.38
2.36	20	44		66.9	33.45	40.14
1.18				43.1	21.55	25.86
0.6				28.7	14.35	17.22
0.425	9	25				
0.3				19.2	9.6	11.52
0.075	4	15		9.4	4.7	5.64

Figure 5.7 Particle size distributions of specified, available, and blended aggregates

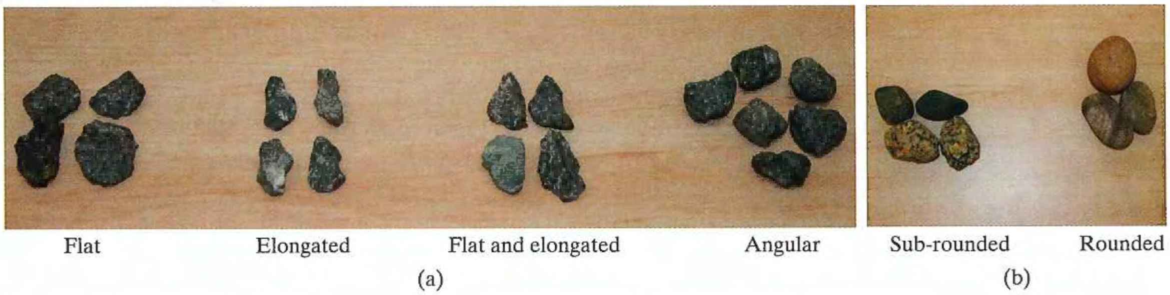


5.3.3 Shape and Surface Texture

As indicated previously, aggregate is classified into coarse- and fine-grained aggregates based on whether the material is retained by or passes through a sieve with opening size 4.75 mm (No. 4 sieve). Individual aggregate particle shape and surface texture are important characteristics that contribute to how the aggregate particles will pack into a dense form. Terms such as angular, flat (or flaky), elongated, and rounded are used for describing the shape of coarse aggregate particles (see Figure 5.8(a)). The relative sharpness of the edges and corners of the aggregate particle is basically used for defining whether the particle is angular or rounded or subrounded (see Figure 5.8(b) for typical examples of rounded and subrounded particles). Moreover, the relative dimensions of the aggregate particle are used for defining the shape of particle as: *flat (or flaky)*, *elongated*, and *flat and elongated* (or *flaky and elongated*). For example, if the thickness is small compared to the length and width then it is referred as *flat or flaky* (see ASTM D4791 for details of the test method for determining the percentages of flat particles, elongated particles, or flat and elongated particles in coarse aggregate). The common terms used for describing the shape, brief description, and typical examples are summarized in Table 5.10.

Typical examples of different particle shapes of coarse aggregate: (a) flat, elongated, flat and elongated, and angular (b) subrounded and rounded

Figure 5.8



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Particle shape classification of coarse aggregate with typical examples (modified from Neville, 2011)

Table 5.10

Classification	Description	Examples
Angular	Well-defined edges formed at the intersection of roughly planer faces; no evidence of wear	Crushed rock (of all types); crushed slag
Flat (or flaky)	Thickness is small compared to the other two dimensions. ASTM defines flat and elongated particles as those having *width to thickness ratio* greater than a specified value	Laminated rock (e.g., shale)
Elongated	Usually irregular in which the length is considerably larger than the other two dimensions. ASTM defines elongated particles as those having *length to width ratio* greater than a specified value	
Flat and elongated (or flaky and elongated)	The length is considerably larger than the width, and the width is considerably larger than the thickness. ASTM defines flat and elongated particles as those having *length to thickness ratio* greater than a specified value	
Rounded	Edges and corners smoothed or worn out by attrition, faces of particles nonplaner.	River bed and seashore gravel; desert, seashore and wind-blown sand
Subrounded	Considerable wear, faces reduced in area	

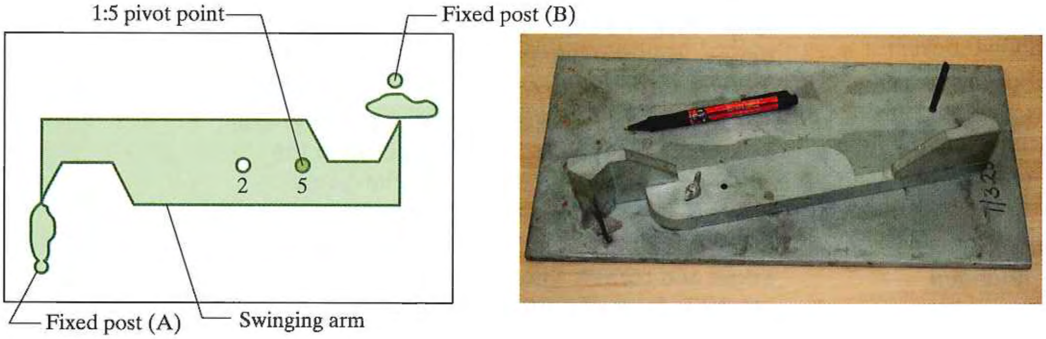
(Based on Neville, A.M. (2011). Properties of concrete. Fifth Edition, Pearson.)

A schematic of the device used for measuring the ratios of the dimensions of the coarse aggregate particle, in particular the length to thickness, length to width, or width to thickness ratio, is shown in Figure 5.9. The device typically consists of a swinging arm and two posts. For example, to determine the ratio between the length and thickness, the particle is placed between the pointed part of the swinging arm and fixed post A as shown in Figure 5.9 first, and then its thickness is checked to see whether the particle could pass through the gap between the other pointed part of the swinging arm and post B while keeping the swinging arm stationary. The device has the capability of determining whether the ratio is closer to either 1:2, 1:3, 1:4, or 1:5 by changing either the position of the pivot point or the pointed part to fixed post combination. Further details of the test apparatus and the method for determining the percentages of flat particles, elongated particles and flat and elongated particles in a coarse aggregate can be found in ASTM D4791.

Surface Texture of Coarse Aggregate

When aggregate is used as an unbound material in applications such as road pavement or foundation construction, it is usually compacted to achieve higher density in order to improve its strength and stiffness. In addition to the shape of individual particles, surface texture is also an important characteristic that determines how the aggregate particles will move around during compaction to pack into a dense form. Strength and stiffness of unbound materials are attained mainly from the interface friction between particles and interlocking of aggregate particles. If the surface texture is rough, the particles will resist movement and consequently be difficult to compact. However, the rough surface texture will improve interparticle friction contributing to improved shear strength of unbound aggregate. Aggregate with flaky and elongated particles are difficult to compact and hence not desirable, whereas aggregate with angular particles is desirable. Strength and stiffness of bound materials are attained mainly from the cementing action or cement-aggregate bonding and interlocking of aggregate particles. When aggregate with particles of rough surface texture are used to make bound materials like PCC or AC, they will have good bonding resulting in improved strength.

Figure 5.9 Device used for measuring the ratio between different dimensions of a coarse aggregate particle: (a) schematic figure of lever arm device (b) photo of lever arm device



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Aggregate with rounded and smooth particles will be easy to compact because of less interparticle friction but display lesser shear strength when used as unbound materials. They improve workability when used in PCC (or AC) but less strength and stiffness. They are less desirable for use in surface course or surface treatment of road pavements due to low friction and hence less surface traction for the vehicles. Aggregate with angular particles are desirable in unbound as well as bound materials. The percentage of fractured particles in coarse aggregates, which are desirable due to rough surface texture, is specified in certain material specifications and hence a standard test method for testing a coarse aggregate (for example, ASTM D5821) has been developed for such purposes.

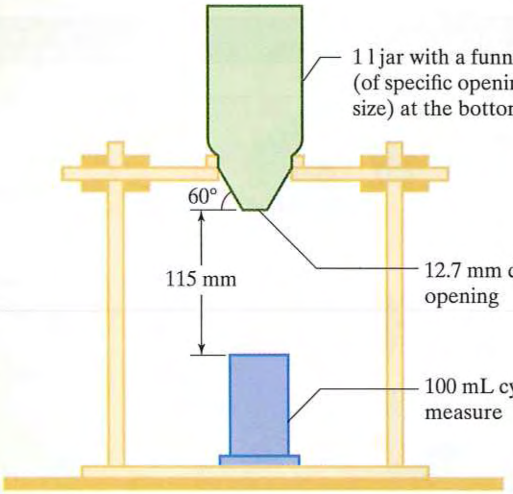
Surface Texture of Fine Aggregate

Surface texture and angularity of fine aggregates are also important as they significantly influence the characteristics or mix design of materials such as AC, PCC, grout, and mortar. However, surface texture and angularity properties of fine aggregates are difficult to be assessed and quantified from measurements of individual particles due to their relatively small size. Hence, the uncompacted void content of a dry sample of the fine aggregate with known grading is assessed, which gives an indication of its angularity and surface texture. A schematic of the apparatus used for determining the uncompacted void content is shown in Figure 5.10. In this test, a dry sample of fine aggregate is allowed to free-fall from a jar with a funnel of standard opening size at the bottom over a specified height (typically 115 mm) into a volume measuring cylinder (volume V mL; typically 100 mL). When the cylinder becomes full, the aggregate is levelled with a straight edge, and the mass of the fine aggregate in the cylinder is measured (mass M g). The uncompacted void content (U) of the fine aggregate is calculated as:

$$\mathrm{U} = \frac{\mathrm{V} - \left(\frac{\mathrm{M}}{\mathrm{G}}\right)}{\mathrm{V}} \times 100\ \% \tag{5.3}$$

Schematic of the apparatus used for determining the uncompacted void content of a fine aggregate

Figure 5.10



where G is the relative density or specific gravity of dry fine aggregate particles. Further details of the uncompacted void content testing of fine aggregates can be found in ASTM C1252.

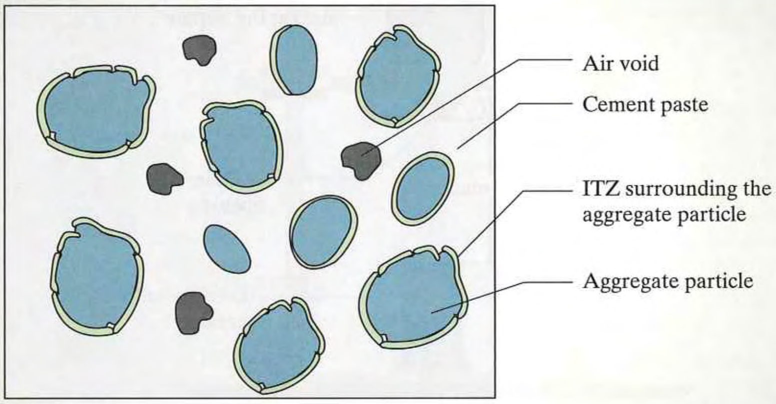
The uncompacted void content information is useful as an indicator of properties such as the mixing water demand of PCC; the flowability, pumpability, or workability of grouts or mortars; the effect of fine aggregate on the stability and voids in mineral aggregate in an AC; the stability of fine aggregate portion of a base course aggregate (ASTM C1252). In particular, the uncompacted void content test result is required when a particular fine aggregate is to be used for making AC in accordance with the Superpave (AC) mix design method. Three different methods are specified in ASTM C1252 (Methods A, B and C) for preparing the fine aggregate test sample. The Superpave AC mix design requires the use of Method A for its fine aggregate component. Fine aggregate having high uncompacted void content will have particles with more fractured faces and hence be desirable for making AC. Typically, Superpave specifies uncompacted void content of at least 45 percent for the fine aggregate in the AC mix for use in high-volume roads (FHWA, 2001).

PCC concrete consists of aggregate particles, cement paste, and the paste region immediately surrounding the aggregate particles, referred as interfacial transition zone (ITZ), as illustrated in Figure 5.11. Thickness of ITZ varies and it is in the order of 30 μm. Properties of PCC are influenced by cement paste, aggregate, and the ITZ. ITZ is structurally weak because of its higher porosity and lower cement content than the bulk cement paste region.

5.3.4 Strength and Stiffness

The strength of the parent rock from which crushed aggregate is produced could be determined from in situ testing (for example, the uniaxial compression test) or testing of core samples. However, the strength and stiffness properties of parent rock may be influenced by factors such as spacing and inclination of joints and imperfections. When the rock is crushed into smaller particles, not all such factors will be relevant, and the strength of aggregate particles could be significantly different from that of parent rock. It is difficult to determine the strength and modulus of elasticity (or stiffness) of individual aggregate particles by testing. However, aggregate is not

Figure 5.11 Schematic illustration of ITZ in PCC



used as individual particles but rather as a collection of particles. Hence, properties of bulk aggregate are generally assessed from tests such as, the aggregate crushing value (ACV) test and resilient modulus from the triaxial test, which are discussed below.

The strength and stiffness of aggregate is dependent on its composition, texture, and structure (Neville, 2011). If an aggregate displays low strength, it may be that the constituents are of low strength or that the grains are strong but they are not well knit together into a competent structure. It is worth noting, however, that the strength of PCC or AC that is made using a particular aggregate cannot significantly exceed the strength of the aggregate which generally constitutes the major part of them.

In the aggregate crushing value (ACV) test, aggregate that passes the 12.5 mm opening size sieve and is retained on 10 mm sieve is tested. A sample of this aggregate is prepared in three layers in a rigid cylindrical rigid mold, measuring 150 mm in diameter and 130–140 mm in height, with a base plate. Twenty-five strokes of tamping are applied for each layer with a 16 mm diameter circular cross section, 600 mm long tamping rod when preparing the sample. The mass of this sample is measured initially (A). A gradual load is then applied through a piston in a compression machine to reach 400 kN (~40 t, ~90,000 lb) over a period of 10 minutes, and then the load is released. Afterward, the sample is sieved though a 2.36 mm opening size sieve, and the mass of aggregate sample that passes through the 2.36 mm opening size sieve (B) is determined. The ACV is defined as: $ACV = \frac{B}{A} \times 100$ percent. Further details of ACV test can be found in BS 812-110 and BS EN 12620.

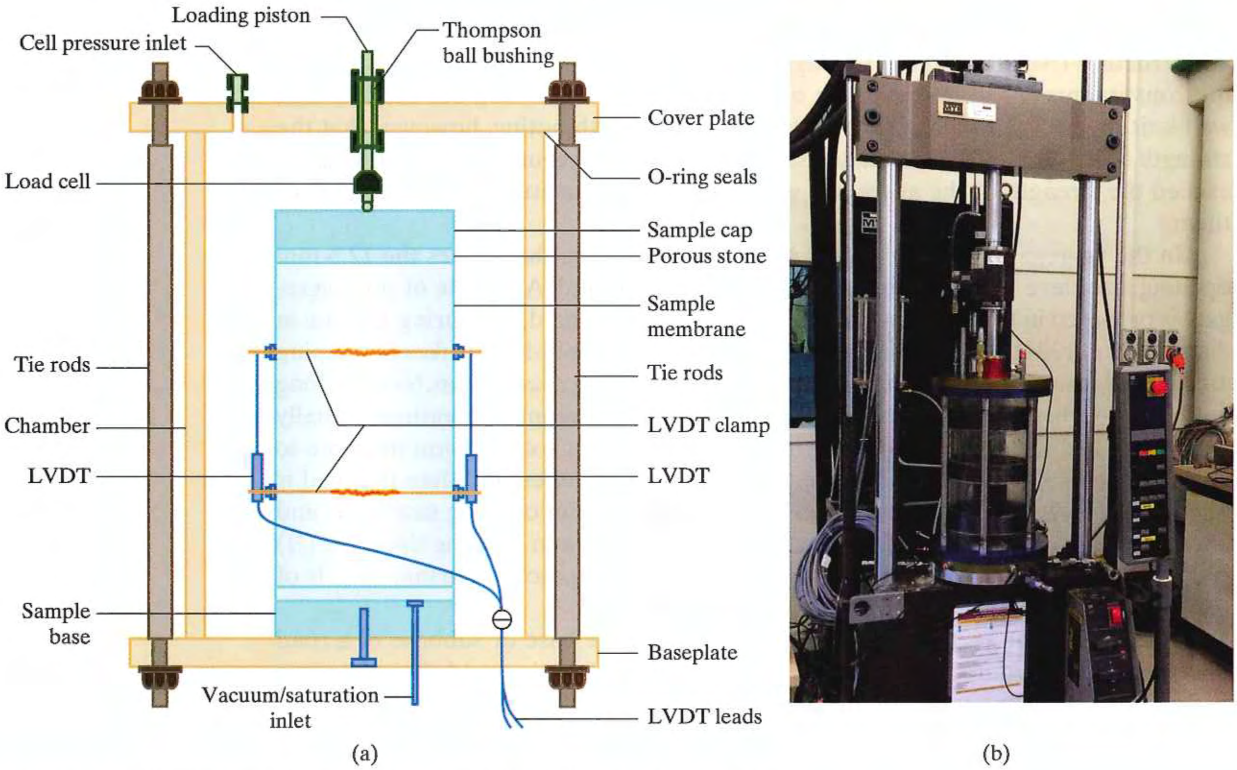
The stiffness of aggregate used for constructing the base or subbase of a road pavement is often determined from cyclic load triaxial testing, and the schematic and a photo of such equipment are shown in Figure 5.12. A cylindrical sample, typically measuring 100 mm diameter and 200 mm height, of aggregate encased in a rubber membrane is subjected to all-round confinement pressure by the fluid (such as de-aired water) in the triaxial cell. In addition, cyclic axial load is applied to the sample to simulate traffic loading on the road, and the axial deformation of the sample is measured using internally mounted LVDTs. The deformation of the sample during a loading cycle consists of two components: namely permanent deformation and elastic (or resilient) deformation (see Figure 5.13). The stress in the sample corresponding to the axial load is referred as deviatoric stress (σ_d). The ratio between the axial deviatoric stress and the strain corresponding to the resilient deformation (ε_a) is referred as resilient modulus (M_D) of the aggregate ($M_D = \sigma_d/\varepsilon_a$). Further details of resilient modulus testing could be found in various standards, for example, AASHTO T292).

5.3.5 Hardness, Toughness, and Abrasion Resistance

Hardness of a material generally refers to its ability to resist shape change. It is measured from scratch or indentation or impact rebound test. The following scratch test guideline (Barksdale, 1991) is used to classify rocks as very hard, hard, moderately hard and soft:

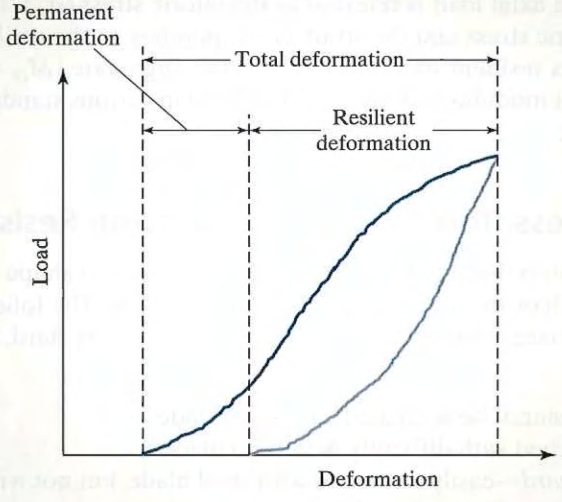
Very hard—cannot be scratched with steel blade
Hard—scratched with difficulty with steel blade
Moderately hard—easily scratched with steel blade, but not with finger nail
Soft—scratches with finger nail

Figure 5.12 Cyclic triaxial testing equipment: (a) schematic (b) photo of typical equipment



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 5.13 Load–deformation behavior during a loading cycle and definition of terms



Resistance of a rock sample to failure due to *impact* is generally defined as *toughness*, but this test is not used for individual aggregate particles. A sample of bulk aggregate with a narrow particle size range is tested in a standard size cylindrical mold, and the effect of impact loading in fragmenting the aggregate particles, referred as aggregate impact value (AIV), is measured through sieving. In particular, aggregate that passes a 12.5 mm opening size sieve and is retained on 10 mm sieve is tested in the AIV test. A sample of this aggregate is prepared in a rigid cylindrical mold with a rigid bottom, measuring 75 mm diameter and 50 mm depth in three layers. Twenty-five strokes of tamping with a 10 mm diameter circular cross section, 230 mm long, tamping rod is applied per layer. The mass of this sample is measured initially (A). Then a 58 kg mass drop-hammer is raised and allowed to free-fall over a height of 380 mm fifteen times. The sample is then sieved through a 2.36 mm opening size sieve, and the mass of aggregate that passes (B) is determined. The AIV is defined as $A/V = \frac{B}{A} \times 100$ percent. Further details of AIV test can be found in BS 812-112: 1990 (2000).

Abrasion resistance of aggregate generally refers to its ability to resist wearing due to abrasion so that the surface texture is retained. This is an important consideration, for example, when assessing the suitability of an aggregate source for use as surfacing material of a road. Certain aggregates (for example limestone) could get polished when used as surfacing material of a road due to traction loading from moving vehicles.

The Los Angeles machine shown in Figure 5.14 is used for testing the resistance to degradation of aggregate by abrasion and impact. Tests have been developed to

Los Angeles machine for abrasion and impact testing

Figure 5.14

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

assess the degradation of aggregate by selecting a sample (~ 5000 g) having specified gradation. A specific number of standard size steel spheres (12 steel balls each weighing about 420 g) are added to the aggregate sample, and the combination is subjected to a certain number of revolutions in the rotating steel drum of the Los Angeles machine. When the drum rotates, a shelf plate fixed on the drum picks up the aggregate sample and steel balls and raises them up. Subsequently they drop (by about 680 mm) to the lower side of the drum thereby creating impact and crushing effect. As the drum rotates further, the sample and steel balls roll on the surface of the drum, which creates abrasion and grinding action on aggregate particles. This cycle of lifting, dropping and rolling of the sample plus steel balls is repeated for prescribed number of revolutions of the drum (typically 500 revolutions), and the resulting aggregate sample is sieved dry over a No. 12 (1.7 mm opening size) sieve. The percent passing on this sieve is referred as the Los Angeles degradation value, sometimes termed as percent loss or percent wear (NSSGA 2013). Further details of the tests for assessing the degradation resistance of coarse aggregate by abrasion and impact can be found in ASTM C535; ASTM C131-06 and AASHTO T-96.

5.3.6 Soundness and Durability

The ability of aggregate to withstand weathering is generally referred as soundness, which is a measure of its durability under environmental (for example, moisture, temperature) and chemical changes. Particles of the aggregate generally contain different minerals and voids in them. Under stark weather changes such as temperature and moisture variations, physical and mechanical properties (like strength and stiffness) of aggregate particles can be affected. For example, an aggregate particle may be subjected to freeze-thaw cycles due to temperature variations under adverse climatic conditions. When the water within the aggregate pores freezes due to changing climatic condition, volume change occurs and this could cause large stresses, which in turn could lead to fracture or disintegration of the aggregate particles. Therefore, soundness is an important consideration for assessing the quality of aggregate for use such as in concrete or other applications where they are subjected to weathering action.

Common tests that are conducted to assess the soundness of aggregate are; wet strength and wet-dry strength variation tests, and sodium sulphate or magnesium sulphate soundness test. The sulphate soundness test involves five cycles of soaking of aggregate sample in saturated sodium or magnesium sulphate solution followed by drying in an oven. Usually, the aggregate sample is sieved and separated into different-sized fractions first, and each fraction sample is immersed in saturated sodium or magnesium sulphate solution for a specified period, typically for 16–18 hours. The sample is then removed and dried completely in an oven until constant mass is attained with the result that sulphate salt precipitates in the pore spaces of the aggregate. This cycle of immersion and drying is repeated five times, during which process aggregate particles get distressed due to stresses induced by a combination of wet/dry and heating/cooling cycles, together with the crystallization pressure from the sodium/magnesium sulphate solution within the pores or defect planes of the aggregate during the drying cycle (Austroads 2008). Many types of distress such as disintegration, splitting, crumbling, cracking, and flaking could take place. The dried aggregate sample is then sieved through the specific opening size sieve, smaller than that on which the sample batch was retained initially, and the percentage loss is determined for each size fraction sample. The weighted average percent loss for each size fraction is used as an indication of the durability of the aggregate (NSSGA 2013). Further details of this testing could be found in ASTM C88–13. Typical limits for the percent loss from soundness test for different classes of aggregates are given in AASHTO specification M 283.

5.3.7 Chemical Stability of Aggregate in PCC

PCC deteriorates with time under normal weathering conditions just as any other material. However, there are certain chemical reactions that may greatly speed up this deterioration, which are of concern for civil engineers and are discussed in this section.

Chemical weathering: Aggregate and the cement paste in PCC could be affected by atmospheric attack such as carbonic acid attack of rainwater or groundwater ($H_2CO_3 \rightarrow H^+ + HCO_3^-$), and sulphurous and sulphuric acid attack of rainwater in areas of SO_2 pollution.

Cement–aggregate reaction: Some deleterious siliceous rocks and minerals in the aggregate react with the Na and K released during the hydration of cement in PCC. This could lead to the formation of swelling gels in the presence of water. Swelling pressures up to 4000 kPa (83.6 kip/ft^2) have been reported in some tests, which is greater than the tensile strength of PCC and hence it cracks. The gels are composed of silica (SiO_2), Na_2O, and K_2O and are watery in the presence of water but brittle white crust when dry, which manifest as white powder from the cracks. There are three common types of cement aggregate reactions, namely alkali-silica reaction (ASR), alkali-silicate reaction, and alkali-carbonate reaction.

Alkali-silica reaction (ASR): This is the most common cement-aggregate reaction, and ASR refers to the reaction between highly alkaline cement paste and silica (i.e., non-crystalline SiO_2) found in many common aggregates. In the presence of moisture, the alkalis found in cement break down the silica in the aggregate, producing an expansive (ASR) gel. This expansion causes tensile forces in PCC, leading to loss of strength or spalling. Some reactive aggregate source rocks and the reactive components in them are listed in Table 5.11.

Alkali-silicate reaction: This refers–the reaction of alkalis from cement paste with silicate minerals found in certain aggregates (for example, aggregates from shaley rocks or aggregates with clay mineral impurities), similar to the ASR discussed earlier but the expansion is relatively slower. Examples of reactive source rocks for this reaction are phyllites, greywackes, and argillites.

Reactive aggregate source rocks and reactive components

Table 5.11

Reactive aggregate source rocks	Reactive components
Opaline cherts	Opal (and chert)
Cherty or chalcedonic limestone	Chalcedony
Siliceous limestone	Chalcedony, chert, opal
Some sandstones	Strained quartz
Some granites	Strained quartz
Rhyolites and rhyolite tuff	Volcanic glass (obsidian)
Glassy intermediate lavas and tuffs	Volcanic glass (obsidian)

Alkali-carbonate reaction: This is a poorly understood, uncommon reaction of alkalis from cement with certain normally stable carbonates such as argillaceous dolomitic limestone. This may be related to dedolomitization, followed by hydration of dry clay minerals.

The remedy for the above cement-aggregate reactions is to use nonreactive aggregate and/or alkali-free cement for the concrete. Also, mix aggregates of different source rocks to dilute reactive components. However, use of certain additives such as fly ash with cement and aggregate for the PCC has also been found to be an effective method.

5.3.8 Cleanness and Deleterious Substances

Aggregate particle surfaces should be clean and free from clay and other particles to develop good bonding with the binder. However, aggregates may contain deleterious substances such as clay, coal, and other friable particles, and limitations are specified ASTM and other standards. One such limitation given in ASTM C33 for concrete fine aggregate is presented in Table 5.12.

5.3.9 Affinity for Asphalt Cement

For strong bonding between the asphalt cement and aggregate in AC, aggregate should have good affinity for asphalt cement. Affinity of aggregate depends on:

- The type of source rock (for example, quartzite, felsite, and rhyolite have poor affinity for asphalt cement)
- Cleanness, shape (angularity), and surface texture of the aggregate
- Dampness of the aggregate, viscosity of the asphalt cement, and the weather

Asphalt cement tends to strip off aggregates of quartzite, felsite, flinty rhyolite, and many types of granite. Special additives are often required to achieve a good bond between the aggregate and asphalt cement.

The rock materials used for asphalt aggregate are generally controlled by the local geology. Some rock types are listed below in the order of value as crushed aggregate for asphalt.

Best—basalts, andesites, diorites, fine-grained gabbros.
Fair—acid volcanics, quartzite, some granite.
Poor—granodiorite, most granites, limestone

Limestone is not very abrasion resistant and tends to polish under traffic, making the pavement slippery when wet. Also, some limestone has very fine pore sizes and is therefore quite susceptible to frost.

Table 5.12 Limits for deleterious substances in fine aggregate for concrete (ASTM C33–13)

Item	Maximum mass percent of total sample
Clay lumps and friable particles; coal and lignite:	3.0
Where surface appearance of concrete is of importance	0.5
All other concrete	1.0

(Based on ASTM C33 - 13)

5.3.10 Moisture and Asphalt Binder Absorption

Water is an essential component for making PCC, and the amount of water used in the concrete-mix influences the workability and strength of PCC. Aggregate particles used for making PCC could contain voids at or near the surface, which could be permeable and absorb moisture. The following terms are used for describing the moisture condition in an aggregate particle:

Oven-dry—No moisture in the aggregate particle
Air-dry—Some moisture in the aggregate particle
Saturated surface–dry (SSD)—Pores and permeable voids of aggregate particle are filled with water but the surface is dry, meaning that there is no free moisture on the surface
Moist—water fills the pores and permeable voids of aggregate particle and the surface is moist, meaning that there is free moisture on the surface

Figure 5.15 illustrates the permeable and impermeable voids in an aggregate particle and the changes that take place when the aggregate particle is oven-dry, air-dry, saturated surface–dry and moist. The moisture absorbed in the aggregate voids may not contribute to the chemical reaction with cement for binding the aggregate particles or to influence the workability of concrete. The amount of absorbed moisture could vary between aggregate types, and it is important to know the amount of absorbed moisture in order to estimate the appropriate amount of water to use for making concrete. This is further discussed in Chapter 8.

The moisture content (MC) of an aggregate is defined as:

$$MC = \frac{\text{Mass of moisture in the aggregate sample}}{\textit{Mass of aggregate} - \textit{solids in the sample}} \times 100$$

The MC of an aggregate is determined as follows:

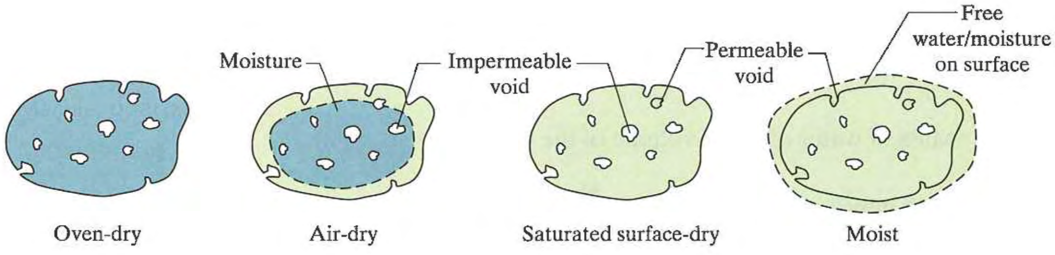
- Weigh an empty container, weight M_c
- Get a representative sample of the aggregate in that container and weigh it, weight M_s
- Leave the sample container in an oven set at the specified temperature (~110°C) and allow sufficient time for the aggregate sample to become completely dry
- Weigh the container with the dry sample, weight M_d

The MC of the aggregate is calculated using:

$$MC = \frac{\mathrm{M_s} - \mathrm{M_d}}{\mathrm{M_d} - \mathrm{M_c}} \times 100 \tag{5.4}$$

Illustration of moisture changes in aggregate

Figure 5.15



EXAMPLE 5.4

An aggregate sample was collected in a container and weighed as 528.7 g. The sample was oven dried and the oven-dry weight was measured as 506.4 g. If the empty weight of the container was 79.6 g, determine the MC of the aggregate.

Solution:

$M_s = 528.7$ g

$M_d = 506.4$ g

$M_c = 76.2$ g

$$\text{MC} = \frac{\text{M}_s - \text{M}_d}{\text{M}_d - \text{M}_c} \times 100 = \frac{528.7 - 506.4}{506.4 - 76.2} \times 100 = 5.2 \text{ percent}$$

Similar to moisture in PCC, the asphalt cement absorbed in the permeable voids of aggregate particles may not be available for binding the particles in AC. The amount of asphalt binder required for an AC mix depends on factors such as the proportion of permeable voids in aggregate, surface texture of aggregate particles, and viscosity of the asphalt binder. These factors are accounted for when determining the volume percentages of different constituents in an AC mix and they are discussed further in Chapter 7.

The volume (or weight) percentages of the constituents in PCC or AC mix are usually calculated on the basis of relative densities (or specific gravities) of the components. The different terms used for the specific gravity of aggregate are discussed in the next section.

5.3.11 Relative Density (or Specific Gravity) and Bulk Density

Relative density (or specific gravity) of a material is defined as the ratio between the density of the material to the density of a reference material (generally water):

$$\text{Specific gravity of material} = \frac{\textit{Density of material } (\rho_{material})}{\textit{Density of water } (\rho_w)}$$

This is typically determined by measuring the mass of a certain volume of the material; determining the mass of gas-free distilled water at specified temperature (23°C or 73.4°F), hereafter referred to as water, of the same volume; and calculating the ratio between the two. For an impermeable solid material, this is often determined by weighing a sample of the material in air (M_a) and weighing the same sample when it is submerged in water (M_s).

Volume of the material (V) is determined using the Archimedes principle as,

$$\text{V} = \frac{(M_a - M_s)}{\rho_w} \tag{5.5}$$

Mass of water of equal volume of the material = $M_a - M_s$.

$$\text{Bulk density of the material} = \frac{M_a}{V} = \frac{M_a}{(M_a - M_s)} \times \rho_w \tag{5.6}$$

$$\text{Bulk specific gravity of the material} = \frac{M_a}{(M_a - M_s)} \quad \textbf{(5.7)}$$

However, as discussed earlier, aggregate particles could contain voids at or near the surface, which could be permeable and absorb moisture. To account for the permeable and impermeable voids in aggregate particles, three types of specific gravities are defined for use in the mix design of PCC and AC.

Apparent specific gravity, G_{sa}

$$G_{sa} = \frac{\textit{Dry mass of aggregate}}{\textit{Volume of aggregate}\text{ solids} + \textit{Volume of impermeable voids}} \times \frac{1}{\rho_w}$$

$$G_{sa} = \frac{M_s}{V_s + V_{miv}} \times \frac{1}{\rho_w} \quad \textbf{(5.8)}$$

$$\textit{Bulk dry specific gravity, } G_{sb} = \frac{\textit{Dry mass of aggregate}}{\textit{Total volume of aggregate}} \times \frac{1}{\rho_w}$$

$$= \frac{M_s}{(V_s + V_{mpv} + V_{miv})} \times \frac{1}{\rho_w} \quad \textbf{(5.9)}$$

$$\textit{Bulk SSD specific gravity} = \frac{\textit{SSD mass of aggregate}}{\textit{Total volume of aggregate}} \times \frac{1}{\rho_w}$$

$$= \frac{M_s + M_{wpv}}{(V_s + V_{mpv} + V_{miv})} \times \frac{1}{\rho_w} \quad \textbf{(5.10)}$$

M_s = mass of aggregate solids

V_s = volume of aggregate solids (excluding permeable and impermeable voids)

V_{mpv} = volume of permeable voids in aggregate

V_{miv} = volume of impermeable voids of aggregate

M_{wpv} = mass of moisture in the permeable voids of aggregate in SSD condition

The apparent specific gravity (G_{sa}) is higher than the bulk dry specific gravity (G_{sb}) since the volume calculation for G_{sa} does not include the moisture permeable voids.

The moisture-permeable voids at or near the surface of aggregate particles that get moist when saturated in water may not get filled entirely by asphalt cement. The above three definitions are therefore not adequate for estimating the asphalt cement quantity in AC. The volume of aggregate voids accessible for asphalt cement, V_{apv}, is an important consideration for AC mix design and hence indirectly included in the calculation of *effective specific gravity, G_{se}*, defined as:

$$G_{se} = \frac{\textit{Dry mass of aggregate}}{\textit{Volume of aggregate solids} + \textit{Volume of voids not accessible to asphalt binder}} \times \frac{1}{\rho_w}$$

$$G_{se} = \frac{M_S}{(V_S + V_{mpv} - V_{apv})} \times \frac{1}{\rho_W} \quad \textbf{(5.11)}$$

where V_{mpv} = volume of permeable voids in aggregate, and

V_{apv} = volume of aggregate voids accessible for asphalt cement

Effective specific gravity (G_{se}) is not directly measured but calculated using the above equation. Details regarding the mass/volume of asphalt cement and aggregate in AC are discussed further in Chapter 7.

Test methods for determining the specific gravity and moisture absorption of coarse aggregate are detailed in ASTM 127, and a brief outline of the procedure and the calculations are given below:

In this method, the mass of a representative oven-dry sample of coarse aggregate is measured (A). Recall that coarse aggregate refers to the material retained on 4.75 mm sieve.

The test sample of coarse aggregate is immersed in water for about 24 hours.

The sample is then taken out, rolled over a moisture-absorbent cloth, and all visible water film from the aggregate particles is removed. The aggregate particles are said to be in saturated surface–dry condition, and its mass (B) is determined.

The saturated surface–dry sample is then immediately immersed in water at 23°C (73.4°F), and its apparent mass (when submerged) in water (C) is determined.

Specific gravities and moisture absorption of the coarse aggregate are calculated as follows:

$$\text{Specific gravity} = \frac{A}{B - C}$$

$$\text{SSD specific gravity} = \frac{B}{B - C}$$

$$\text{Apparent specific gravity} = \frac{A}{A - C}$$

$$\text{Absorption} = \frac{B - A}{A} \times 100\ \%$$

Where

A = mass of oven-dry sample in air

B = mass of saturated surface-dry in air

C = apparent mass of saturated surface-dry in water

The test method for determining the specific gravity and moisture absorption of fine aggregate is detailed in ASTM 128, and it is briefly outlined here:

- A representative sample (~500 g) of fine aggregate that passes a 4.75 mm sieve is immersed in water for 24 hours to saturate the pores.
- The sample is then removed from the water, surface dried to SSD condition, and its mass determined following the procedure detailed in ASTM 128.
- The volume of the sample is determined by placing it in a pycnometer, a constant volume flask, and adding water to the known volume mark (for example, 500 cm^3). The mass of the pycnometer with the sample and water is determined.
- The sample is then poured out and dried in an oven and its oven-dry mass determined.
- The mass of the pycnometer filled with water to the known volume mark is also determined.

Specific gravities and moisture absorption of the fine aggregate are calculated as follows:

$$\text{Specific gravity} = \frac{A}{B + S - C}$$

$$\text{SSD specific gravity} = \frac{S}{B + S - C}$$

$$\text{Apparent specific gravity} = \frac{A}{B + A - C}$$

$$\text{Absorption} = \frac{S - A}{A} \times 100 \text{ percent}$$

A = mass of oven-dry sample

B = mass of pycnometer filled with water

C = mass of pycnometer filled with fine aggregate sample and water

S = mass of the sample at SSD condition

The specific gravities and moisture absorption are used in PCC mix design, and the details are given in Chapter 8.

5.4 Uses of Aggregates

Aggregates are widely used for constructing the base, subbase, and/or the surfacing/wearing course of road pavements in different forms such as:

- Unbound (that is, no cement or other binder)
- Stabilized, using portland cement as the stabilizer for the aggregate
- Cementitiously stabilized aggregates in which blends of cement, fly ash, slag, and lime are used as the stabilizer
- Stabilized with bituminous materials (bitumen or tar are used as the stabilizer/binder) as road base/subbase
- Stabilized with other materials (resins, fibers, geosynthetics, etc.) as road base/subbase
- Recycled aggregate

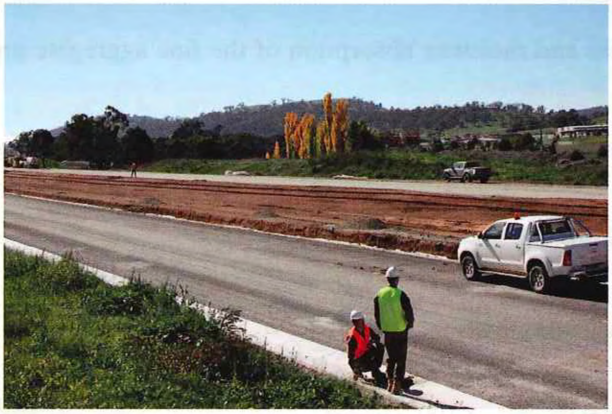
As discussed in various sections previously, aggregates are the major constituent in concrete. Typically 60–75 percent by volume and 79–85 percent by weight are made of aggregates in portland cement concrete.

There are other uses of aggregates in civil engineering projects as:

- Fills, backfills, and other applications
- Drainage and filtration application

Typical examples of aggregate use in civil engineering construction are shown in Figures 5.16–5.18. As indicated earlier, aggregate is an important constituent of PCC which is commonly used in the construction of various civil engineering structures.

Figure 5.16 Completed unbound aggregate road base



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 5.17 Unbound aggregate drainage layer with a geotextile filter for a road embankment

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

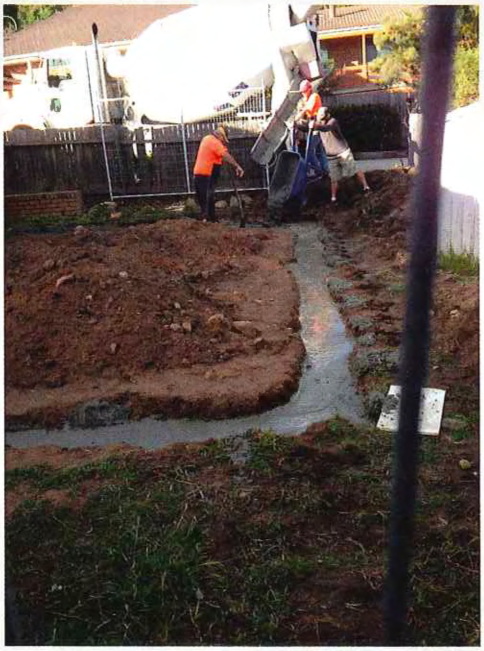
Figure 5.18 Subsoil drain for a road, using aggregate and geotextile filter

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Examples of concreting for a house wall-foundation and a pile-foundation are shown in Figures 5.19 and 5.20, respectively. Typical steel-reinforced concrete retaining structures being constructed for a light-rail subway are shown in Figure 5.21.

Concreting for the wall foundation of a house

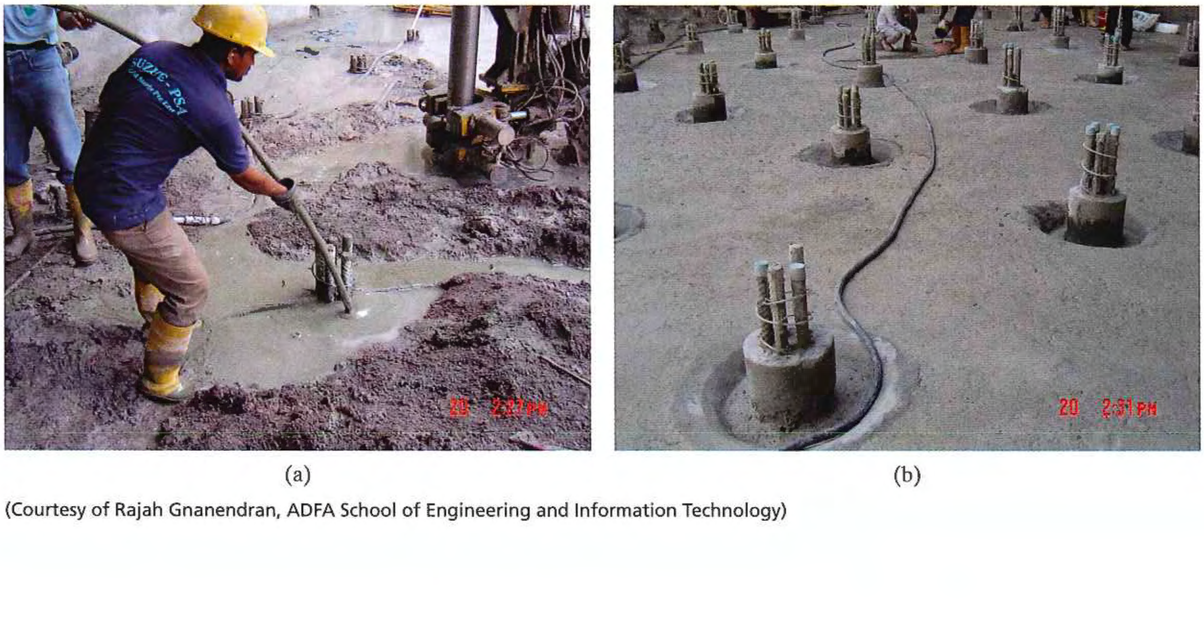
Figure 5.19



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Steel-reinforced concrete mini-pile foundation of a building:
(a) Concreting for a mini-pile; (b) Partially completed mini-piles

Figure 5.20



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 5.21 Construction of steel-reinforced concrete retaining wall structures

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

5.5 Lightweight and Heavyweight Aggregates

Certain manufactured or synthetic lightweight aggregates are also used in civil engineering construction. Typical examples are crushed air cooled blast furnace slag; lightweight aggregates from expanded slates and fly ash that are used for making lightweight concrete; lightweight concrete masonry units and lightweight fill material for constructing certain road embankments. Photos of typical lightweight aggregates used for making lightweight concrete are shown in Figure 5.22. Lightweight aggregates are also produced by processing certain natural materials such as pumice,

Figure 5.22 Typical (a) 8 mm, and (b) 14 mm lightweight aggregates

(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

scoria, or tuff. Lightweight is due to the cellular or high internal porous microstructure that gives the low specific gravity for this type of aggregate. They are porous and have high absorption values. Specification for lightweight aggregates for structural concrete where the main considerations are reducing the density while maintaining the compressive strength of concrete is detailed in ASTM C330. Use of lightweight aggregates for concrete masonry units when the main consideration is to reduce the density of the units is covered in ASTM C331.

Heavyweight aggregates are typically natural mineral aggregates such as barite, magnetite, or limonite and manufactured aggregates such as iron, lead, or steel shots used for producing high-density concrete. The main applications of heavyweight concrete are radiation shielding in medical or nuclear fields and ballasting for off-shore pipelines.

5.6 Aggregates from Industrial By-Products and Waste

Air-cooled blast furnace slag (ACBFS) aggregate: Blast furnace slag is a byproduct obtained during the production of steel from pig iron in a blast furnace. ACBFS is produced through relatively slow solidification of molten blast furnace slag under atmospheric conditions. Crushed ACBFS is one of the most commonly utilized reclaimed construction materials, being used as coarse aggregate in PCC, aggregate in HMA, road base material, and fill (FWHA 2012). Approximately 6.9 million metric tons (7.6 million tons) of ACBFS were used in the United States in 2008, about 39 percent of it as road base or in road surface layers, 17 percent in asphalt concrete, 17 percent in ready-mixed concrete, 9 percent as fill, 4 percent in the manufacture of concrete products, and the rest was used in miscellaneous applications such as railroad ballast and roofing (USGS 2009). Although this accounts for only 2 percent of the aggregate used for construction in the United States, ACBFS aggregate use in construction around the world is growing.

ACBFS aggregate is a crushed product with angular, roughly cubical particles having pitted vesicular surfaces. Compared to naturally derived aggregate, ACBFS aggregate has:

- higher porosity and absorption,
- low specific gravity,
- high Los Angeles abrasion loss, and
- high sulphate and sulphur content.

Absorption is a concern particularly when ACBFS aggregate is batched dry since it could affect the workability and shrinkage potential of PCC. Therefore, it is commonly kept sufficiently moist prior to batching when used in PCC.

Also, ACBFS has potential iron unsoundness, dicalcium silicate unsoundness, and presence of free lime (CaO) that require periodic chemical testing (FWHA 2012). Iron unsoundness is rare, and it arises only if partially reduced iron oxides are present. If present, they get oxidized due to the expansive reaction, causing the ACBFS to disintegrate. Iron unsoundness is commonly tested by immersing pieces of ACBFS in water for fourteen days and detecting crack or disintegration in the particles.

Dicalcium silicate unsoundness arise if beta-form dicalcium silicate is present in ACBFS that inverts to the stable gamma-form, resulting in volume increase, after ACBFS aggregate is used in PCC. As discussed earlier, ACBFS is produced by slow cooling and solidification of molten blast furnace slag, and the dicalcium silicate phase inversion is expected to take place during the cooling process. The potential dicalcium silicate unsoundness is mitigated by allowing the ACBFS aggregate stockpiles to weather for extended periods before being used in PCC.

When used in PCC, calcium sulphide in the ACBFS aggregate becoming more soluble in the high alkali environment of cement paste is considered a major risk. This could lead to potential freeze-thaw attack or an internal sulphate attack or both. European standards have imposed a 2 percent limit on the total sulphur content of ACBFS aggregate used in PCC.

Although national standards are not yet available in the United States, local standard specifications have been developed by state highway departments for ACBFS aggregate's use in paving concrete, HMA for base/surface-course, and as unbound material for the base/subbase of pavements. However, there are international standards such as the European Standard, *CEN EN 12620—Aggregates for Concrete*; Australian Standard, *AS 2758.1-1998—Aggregate and Rock for Engineering Purposes—Part 1: Concrete Aggregates*; Japanese Industrial Standard *JIS A 5011-1977—Air-cooled blast furnace slag aggregate for concrete*, and *JIS A 5011-1:2003—Slag aggregate for concrete—Part 1: Blast furnace slag aggregate* for the use of ACBFS aggregate in PCC.

Recycled aggregate: Diminishing sources of high-quality natural aggregates, quarrying and transportation costs, and environmental pressures have resulted in the search for alternatives. On the other hand, the need for demolition and renewal of buildings, roads, and other infrastructure is continuing to grow. Moreover, there is emphasis to innovate wherever possible to provide maximum value for limited resources and funds available for construction. A solution to these multifaceted problems is to utilize the recycled aggregate and clean fill materials obtained from such demolition and renewal, as outlined in the schematic shown in Figure 5.23 for constructing roads, buildings, and other structures. Such reuse is also beneficial by reducing the amount of materials from demolition to be disposed in places like landfills. There are two main sources for such recycled aggregate, namely aggregate from *reclaimed asphalt pavement* (RAP) and *building demolition waste.*

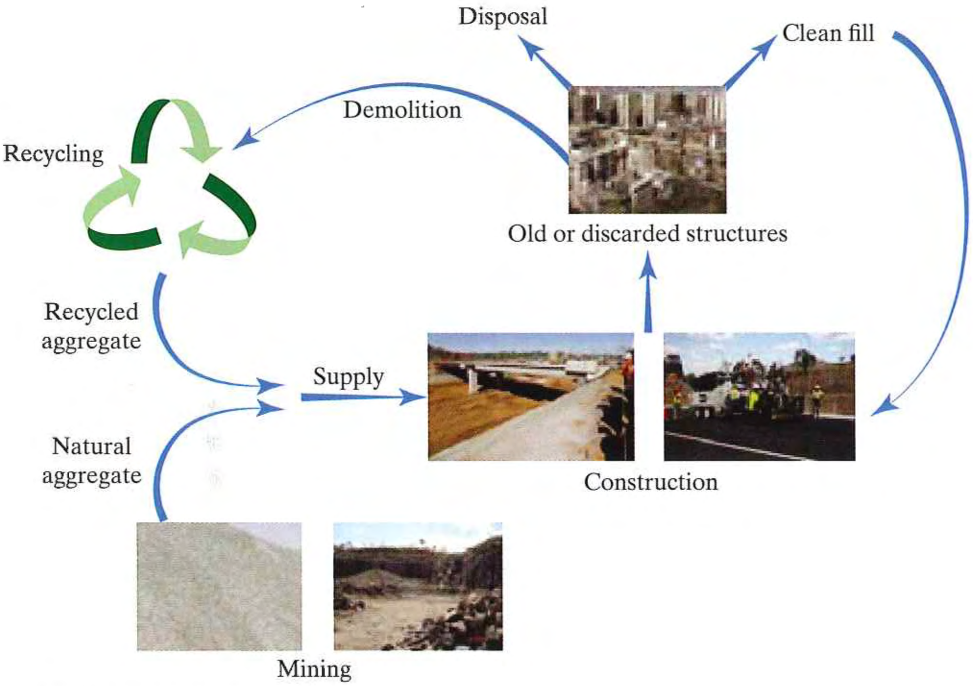
RAP: Over 90 percent of the highways and roads in the United States are built with HMA, and they are maintained and rehabilitated when they age (FHWA 2011). The HMA is often milled or completely removed and replaced with new HMA during the maintenance and rehabilitation, and the material is processed for reuse. This processed material, commonly called RAP, is mainly used for making HMA by blending it with virgin aggregate. The processing involves crushing and screening to produce uniform gradation, asphalt cement content, and other properties. The main concern for using RAP material in HMA for a road rehabilitation or construction project is product variability. RAP originating from different road rehabilitation projects is a main contributing factor for the variability.

The use of RAP in HMA is estimated to account for 10–20 percent of the total HMA produced in the United States. The asphalt cement coating on the RAP material has the beneficial effect of reducing the asphalt cement binder required for the HMA. HMA mix design involving the use of RAP is performed in accordance to the guidelines given in AASHTO M 323.

Aggregate from building demolition waste: Building demolition waste is generally crushed and screened similar to RAP before reuse. The resulting aggregate, also referred to as crushed concrete, could be used in PCC or in the construction of road

Schematic of recycled aggregate production

Figure 5.23



(Modified from USGS 1998)

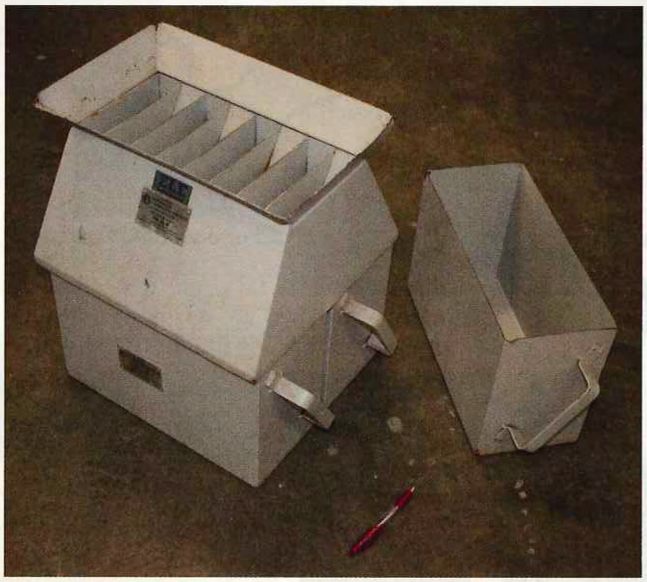
base/subbase as either cement-stabilized or unbound material. Sometimes, it is used as the fill material such as the general fill for constructing road embankments. This recycled aggregate could also be blended with virgin aggregate to satisfy the specification requirements such as gradation and density for a particular application.

5.7 Handling, Transportation, and Storage of Aggregates

Aggregate used in civil engineering construction generally consists of particles of different sizes and shapes. Aggregate particles tend to segregate such that bigger size particles move to the bottom and the smaller ones get near the surface, when they roll down the slope of a stockpile or when tipped off a transporting truck. Segregation could be minimized by stockpiling in thin layers and wherever possible moving aggregates using a belt fitted with a baffle plate at the end to remix the aggregates.

Getting representative samples is vital for determining the properties of the aggregate. A riffle sample splitter shown in Figure 5.24 is often used for splitting dry aggregate samples into small batches for carrying out various tests. Details regarding sampling and reducing large samples of aggregate for laboratory testing can be found in ASTM D75, AASHTO T2, ASTM C702, and AASHTO T248.

Figure 5.24 Riffle boxes for splitting dry samples into two small batches



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

5.8 Summary

1. Aggregate generally refers to a collection of disparate elements, size ranging from <0.1 mm to > 50 mm, of particulate rock-like material. Aggregate material retained on 4.75 mm opening size sieve is referred as *coarse-grained aggregate*, and that which passes through 4.75 mm sieve is called *fine-grained aggregate*.
2. Aggregates are commonly sourced naturally from gravel pits, river deposits, and rock quarries. Certain manufactured or synthetic aggregates, either lightweight or heavyweight, are also used in civil engineering construction. In addition, recycled or reclaimed aggregates, such as crushed concrete or building demolition waste and reclaimed asphalt pavement, are also used in some construction work.
3. Crushed rocks are the most common aggregates used in civil engineering construction, and their properties are dependent on the source rocks, which are classified in accordance to their geologic origin as igneous, metamorphic, or sedimentary. Aggregates from all three classes of rocks are used in civil engineering construction for different applications.
4. Particle size, shape, surface texture, specific gravity, soundness, asphalt cement affinity, reactivity with chemicals, compressive strength, impact resistance, and abrasion resistance are the properties of aggregates assessed for determining the suitability of aggregate. Relative importance of each of these properties depends on the particular civil engineering application.
5. Aggregates are used as unbound materials (for example in road base construction or as filter/drainage layer) and bound materials such as portland cement concrete (or concrete) and AC (or HMA).

6. Particle size distribution or grading is a common property of aggregate and it is determined from sieve analysis. Grading requirement specifications are given in ASTM and other standards for different applications, for use as unbound or bound material. Aggregates not conforming to the grading specification could be mixed or blended in certain circumstances to satisfy the specifications.
7. Resilient modulus and accumulation of permanent deformation are important properties used for road pavement design involving unbound aggregate, and they are determined by cyclic load triaxial testing.
8. Moisture absorption in aggregate, particularly the optimal amount of water needed for a concrete mix, is an important consideration for concrete mix design. To facilitate this consideration, different specific gravities, namely, apparent specific gravity, bulk dry specific gravity, and bulk saturated surface–dry (SSD) specific gravity, are defined and used.
9. In addition to the above specific gravities, the effective specific gravity is defined for use in AC or HMA mix design.

Exercises

1. What are main sources of aggregates? Name three common uses of aggregates.
2. A sieve analysis test was conducted on a representative sample of a concrete aggregate material for determining its gradation, and the results are given below. Calculate the percent passing through each sieve and plot the grading curve of the aggregate.

   The following size sieves were used and the net weights of the material retained on each of them are:

Sieve size (mm)	12.5	9.5	4.75	2.36	1.18	0.60	0.425	0.30	0.15	0.075	Pan
Amount retained (g)	0	38.7	36.8	77.5	127.8	164.6	174.3	141.4	105.6	67.8	33.9

3. Calculate the percentage passing for each common sieve size and plot the maximum-density grading curve of aggregate that has a maximum particle size of 12.5 mm (1/2 in.) in accordance with FHWA recommendation. Compare this with the grading determined in Question 2.
4. A sieve analysis test was conducted on a recycled aggregate sample of a road base material for determining its gradation, and the results are given below. Calculate the percent passing through each sieve and plot the grading curve of the aggregate.

   The following size sieves were used, and the net weights of the material retained on each of them are:

Sieve size (mm)	50.0	37.5	25.0	19.0	12.5	9.5	4.75	Pan
Amount retained (g)	0	107.4	161.6	190.9	235.4	227.8	26.0	1.1

5. Calculate the percentage passing for each common sieve size and plot the maximum-density grading curve of aggregate that has a maximum particle size of 50 mm (2 in.) in accordance with FHWA recommendation and compare with the grading curve of the material plotted in Question 4.
6. What is the significance of different shapes of aggregate particles? Define four different shapes.
7. Discuss the desirable characteristics of aggregates for use in concrete.
8. Discuss the desirable characteristics of aggregates for use in asphalt.
9. What is the significance of soundness of aggregate particles, and describe how it is assessed.
10. Describe what is meant by ASR and the conditions in which this could occur. Briefly outline how the occurrence of ASR could be avoided or its adverse effects minimized.
11. Define the term saturated surface–dry condition of aggregate and discuss how the moisture content of coarse aggregate in saturated surface–dry condition is determined.
12. Define the terms bulk dry specific gravity and apparent specific gravity. Discuss the differences between these two specific gravities.
13. Discuss what is meant by absorbed moisture in aggregate and how it is determined.
14. Discuss the importance of grading of aggregates and describe the different types of grading.
15. Shown in the table below are the specified grading for a road base material and the gradings of the two aggregate materials A and B that are available. Determine the proportion that could be adopted for blending the two aggregates A and B to satisfy the specified grading.

	Percent passing (by mass)		
Sieve size (mm)	Specified grading	Material A	Material B
50	100	100	
37.5	85–100	89	
25	70–90	74	
19	60–80	60.4	100
12.5	48–72	46	96.2
9.5	42–66	38.5	91.5
4.75	28–55	20	76.1
2.36	20–44	7.1	62
0.425	9–25	0.1	32.1
0.075	4–15	0	16.4

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CHAPTER

6

A geogrid type geosynthetic is placed beneath a rail track in Queensland, Australia for stability. Courtesy of N. Sivakugan, James Cook University, Australia

Geosynthetics

6.1 Introduction

Different materials are used for constructing civil engineering structures, and *geosynthetics* are a more recent addition. Geosynthetics are mainly planar, man-made products of polymeric materials used with soil, rock, earth, asphalt, and other geotechnical engineering materials, generally as an integral part of a civil engineering structure (ASTM D4439).

Initially geosynthetics were used as reinforcement for soil, rock, and earth and to facilitate construction in difficult ground conditions. In fact, the practice of introducing "foreign" materials into soil to improve its qualities dates back to at least 3000 BC, when split logs were used to create corduroy roads over peat bogs in Britain. The first use of a fabric to reinforce roads was attempted in 1926, whereby the South Carolina Highway Department used a heavy cotton fabric onto which hot asphalt was applied. However, it was not until the 1970s that the technology was known around the world and perhaps the mid to late 1980s before it was widely accepted. The development and use of geosynthetics in civil engineering applications have rapidly increased over the past three decades due to wide acceptance of these materials and the desire to provide better solutions to expensive, difficult, or previously impossible problems.

There are several types of geosynthetics, and they are commonly used to enhance the stability and/or performance of civil engineering structures such as retaining walls, road bases, road pavements, road embankments, waste disposal facilities (such as landfills), and canals. When used in such an application, a geosynthetic contributes by performing one or more of the following functions: reinforcement, separation, filtration, drainage, erosion control, and containment. Details of the types of geosynthetics, materials used for their manufacture, and their properties, functions, and mechanisms in typical civil engineering applications are examined in this chapter.

6.2 Types of Geosynthetics, Polymers Used, Manufacture, and Common Use

6.2.1 Types of Geosynthetics

Different types of geosynthetics are manufactured for a wide variety of civil engineering applications, and the common types are geotextiles, geogrids, geonets, geomembranes, geocells, geomats, and geocomposites. As indicated earlier, a geosynthetic is intended to perform one or more of the functions of containment, filtration, drainage, separation, reinforcement, and erosion control when used in a civil engineering structure. Geosynthetics are manufactured from different polymers and hence their properties and suitability for use in a particular situation are dependent on the polymer used and the adopted manufacturing process.

6.2.2 Polymers Used in Geosynthetics

Polymers commonly used in the manufacture of geosynthetics are: polyethylene (PE), polypropylene (PP), polyvinyl chloride (PVC), polyester (polyethylene terephthalate—PET), polyamide (PA—nylon 66), and polystyrene (PS). However, other synthetic materials made of aramid and glass are also used in certain civil engineering projects (for example, wrap-around aramid fabric is used as reinforcement in the repair of damaged columns of old buildings). Common polymeric materials, repeating molecular unit of their chemical structure, and the types of geosynthetics that are commonly produced from each polymer are summarized in Table 6.1.

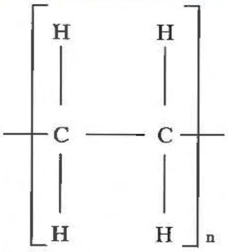
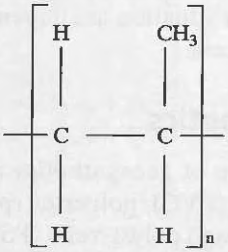
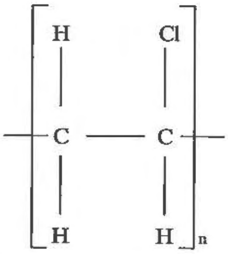
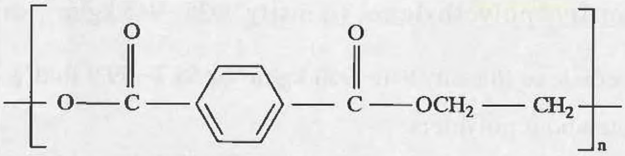
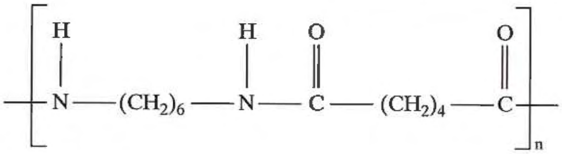
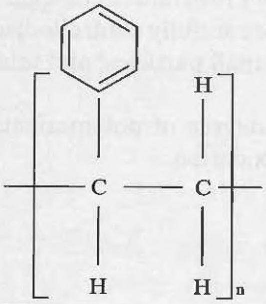
PE is further divided into three groups depending on the density and chemical structure, namely:

- LDPE—low-density polyethylene (density 920–930 kg/m^3 or 57.6–58.0 lb/ft^3)
- LLDPE—linear low-density polyethylene (density 925–945 kg/m^3 or 57.7–59.0 lb/ft^3)
- HDPE—high-density polyethelene (density 940–960 kg/m^3 or 58.7–59.9 lb/ft^3)

A few significant points to note about polymers:

- HDPE is more rigid, stronger, and tougher and has better chemical resistance than the low-density types of PE.
- PP is much more susceptible to oxidation than PE. However, the right choice and quantity of additives can protect PP against aging.
- Additives are used in PET to increase the speed of polymerization, reduce thermal degradation, and increase ultraviolet (UV) resistance. For quality assurance of PET, the degree of polymerization must be carefully controlled; must not be contaminated by dust, small fibers, or other small particles; and must be absolutely dry (moisture content $<0.004\%$).
- Additives are used in PA to control the degree of polymerization, protect the polymer against light, and resist thermo-oxidation.

Table 6.1 Repeating molecular units of polymers used in the manufacture of different types of geosynthetics

Polymer	Repeating unit	Types of geosynthetics
Polyethylene (PE)	 $\left[-\overset{\mathrm{H}}{\underset{\mathrm{H}}{\mathrm{C}}}-\overset{\mathrm{H}}{\underset{\mathrm{H}}{\mathrm{C}}}- \right]_n$	Geotextiles, geomembranes, geogrids, geopipe, geonets, geocomposites
Polypropylene (PP)	 $\left[-\overset{\mathrm{H}}{\underset{\mathrm{H}}{\mathrm{C}}}-\overset{\mathrm{CH_3}}{\underset{\mathrm{H}}{\mathrm{C}}}- \right]_n$	Geotextiles, geomembranes, geogrids, geocomposites
Polyvinyl chloride (PVC)	 $\left[-\overset{\mathrm{H}}{\underset{\mathrm{H}}{\mathrm{C}}}-\overset{\mathrm{Cl}}{\underset{\mathrm{H}}{\mathrm{C}}}- \right]_n$	Geomembranes, geocomposites, geopipe
Polyester (polyethylene terephthalate) (PET)		Geotextiles, geogrids
Polyamide 6.6 (PA) (Nylon 6/6)	 $\left[-\overset{\mathrm{H}}{\mathrm{N}}-(\mathrm{CH_2})_6-\overset{\mathrm{H}}{\mathrm{N}}-\overset{\mathrm{O}}{\overset{\Vert}{\mathrm{C}}}-(\mathrm{CH_2})_4-\overset{\mathrm{O}}{\overset{\Vert}{\mathrm{C}}}- \right]_n$	Geotextiles, geocomposites, geogrids
Polystyrene (PS)		Geocomposites, geofoam

(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

6.2.3 Geotextiles

Geotextiles are the largest group of geosynthetics, and they are in fact textiles that you are familiar with but manufactured using synthetic materials rather than natural materials such as cotton, silk, and wool. Geotextiles are permeable fabrics, generally manufactured from fibers of polymers, such as polypropylene, polyester (polyethylene terephthalate), polyamide (nylon), and polyethylene, using textile technology. However, other polymers, glass, and even steel are also used to a lesser extent.

The range of geotextile products is enormous, as are their applications. Geotextiles are porous but to a widely varying degree, with different products performing at least one of the following four major functions: reinforcement, separation, filtration, and drainage. They are described in the next section. Depending on the method of manufacture, they are classified as woven and nonwoven geotextiles, and a few samples of each group are shown in Figure 6.1.

There are three basic types of synthetic fibers, namely continuous (or mono) filaments, staple fibers, and slit films.

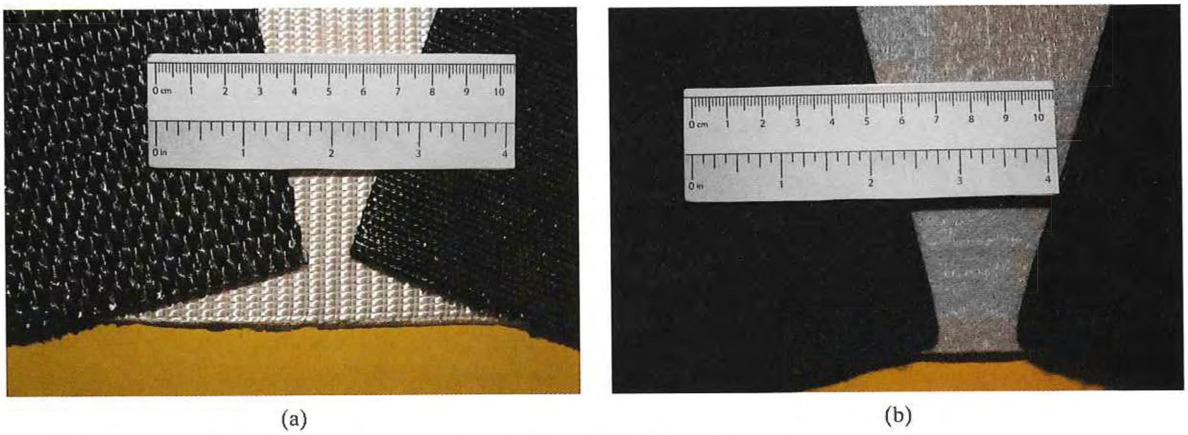
Continuous (or mono) filaments—These are produced by extruding melted polymers. After extrusion, a filament is usually stretched to reduce the fiber diameter and to cause the molecules to arrange themselves in a more orderly fashion. This increases the strength of the filaments, decreases their elongation at failure, and increases their modulus. These filaments, known as *monofilaments*, can also be twisted together to form a *multifilament* yarn.

Staple fibers—Staple fibers are produced by crimping and cutting large bundles of monofilaments into short lengths, typically 25–100 mm (1–4 in.) long, and then twisting or spinning these into long yarns.

Slit films—Slit films are tape-like fibers, typically 1–3 mm (0.04–0.12 in.) wide, that are made by cutting a continuous sheet of polymer. The resultant *slit-film monofilaments* can also be twisted together to form a *slit-film multifilament*

The different types of fibers and yarns, namely monofilament, multifilament, staple fibers, staple yarn, slit-film monofilament, and slit-film multifilament fibers used for manufacturing geotextiles, are illustrated in Figure 6.2. The fibers, or yarns, discussed above are manufactured into fabrics that are collectively known as geotextiles. Some basic properties of synthetic fibers used in the manufacture of geotextiles are presented in Table 6.2.

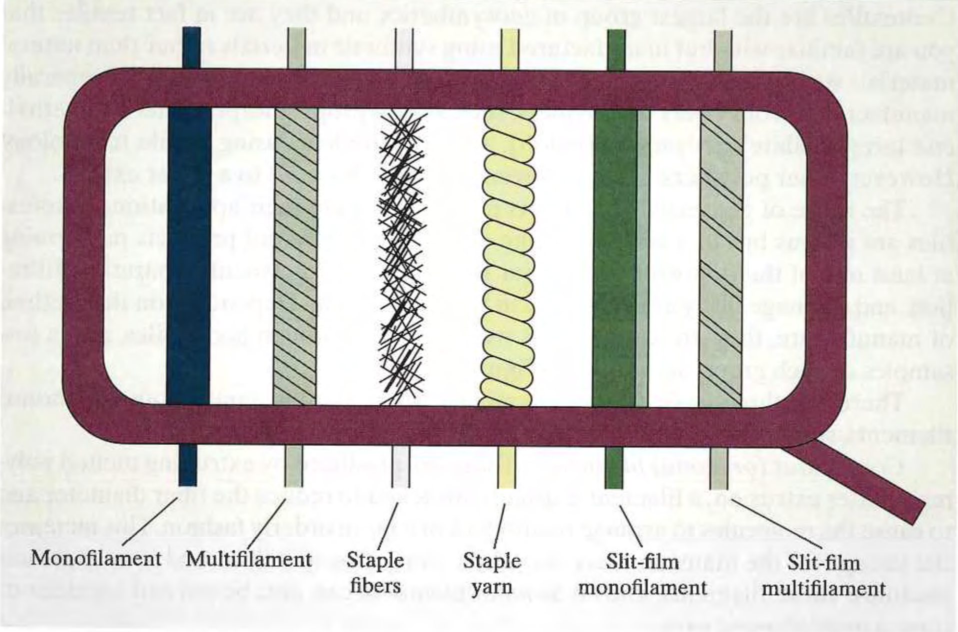
Examples of geotextiles: (a) woven geotextiles (b) nonwoven geotextiles Figure 6.1



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 6.2 Types of polymeric fibers or yarns used for manufacturing geotextiles



(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

Table 6.2 Some properties of synthetic fibers[a]

Fiber type	Breaking tenacity[b] (g/denier)		Specific gravity	Standard moisture regain[c] (%)	Extension to break (%)
	Standard	Wet			
Polyethylene—high density	—	—	0.96	2.0	20–30
Polypropylene	4.8–7.0	4.8–7.0	0.91	3.0	17–20
Polyester—regular tenacity	4.0–5.0	4.0–5.0	1.22–1.38	0.4–0.8	6–30
Polyester—high tenacity	6.3–9.5	6.2–9.4	1.22–1.38	0.4–0.8	6–30
Polyamide Nylon 66—regular tenacity	3.0–6.0	2.6–5.4	1.14	4.0–4.5	12–30
Nylon 66—high tenacity	6.0–9.5	5.0–8.0	1.14	4.0–4.5	12–30

[a] *Standard laboratory conditions for the tests: 21°C (70°F) and 65% relative humidity*

[b] *Breaking tenacity: Stress at which the fiber breaks expressed in grams per denier (A denier is the weight in grams of 9000 m of yarn)*

[c] *Standard moisture regain: The moisture regain of a fiber (expressed as a % of the moisture-free weight)*

(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

As indicated earlier, common methods of manufacture leads to geotextiles being classified as woven and nonwoven. Woven geotextiles or fabrics are basically simple structures of interlacing filaments or yarns. They are usually aligned in two perpendicular directions, namely the longitudinal (or warp) direction and crosswise (or fill or weft) direction. Woven fabrics are typically manufactured using a loom, and its details are illustrated in Figure 6.3. The loom consists of warp threads (fibers or yarns) wound on a beam that are passed through a reed. The reed is capable of separating a predetermined pattern of yarns (for example, alternate yarns) when it is moved up or down as illustrated in Figure 6.3. The shuttle containing the pirn of weft thread (or yarn) is made to go through the gap (or tunnel) to the other side while the reed is up, shedding the weft yarn. The reed is then moved down, taking another (or alternate) set of warp yarns with it, creating another tunnel. The shuttle is then moved back across to the previous side, allowing the interlocking of warp and weft yarns in a specific predetermined pattern. This type of alternate movements of reed and shuttle facilitates the interlacing of warp and weft yarns that result in the manufacturing of the fabric.

As is discussed later, tensile strength and stiffness are important engineering properties of a geotextile, particularly for reinforcement function applications. To cater to wide ranging practical needs, woven geotextiles are often manufactured with a greater tensile strength in the warp direction (also referred as machine direction [MD]) than the fill (or weft) direction (also referred as cross-machine direction [CMD]).

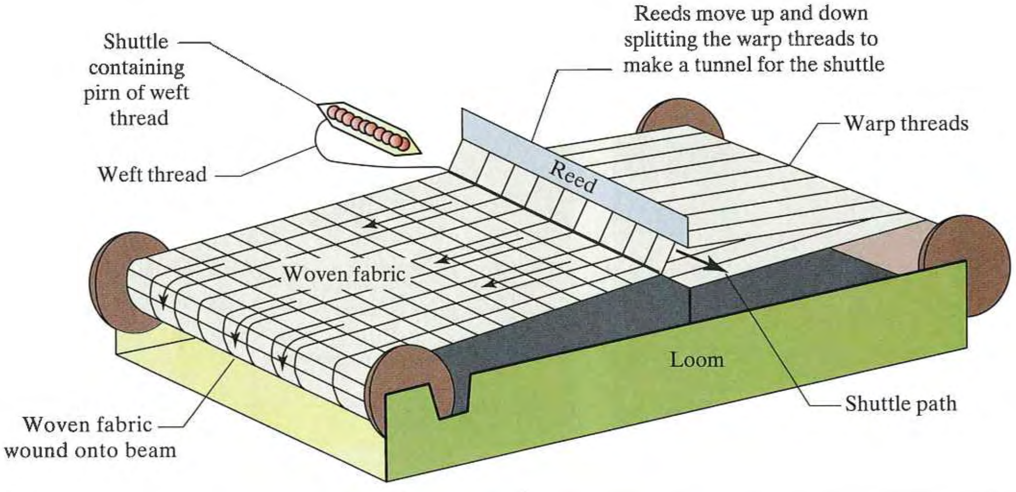
Nonwoven fabrics are formed by spreading filaments onto a conveyor belt and linking them together by one of the following bonding processes:

a. Heat bonding, where filaments are heated to near melting point and subsequently bonded wherever they cross one another. This may also be referred to as thermal bonding or melting.

b. Needle punching, where filaments are bonded by repeatedly driving barber needles through the filament mat, entangling the filaments. This process is the most common and may also be referred to as mechanical bonding.

Details of a weaving loom

Figure 6.3



(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

c. Resin bonding, where the geotextile is sprayed or impregnated with a setting resin. This process can also be used for additional bonding and dimensional stability for needle-punched geotextiles.

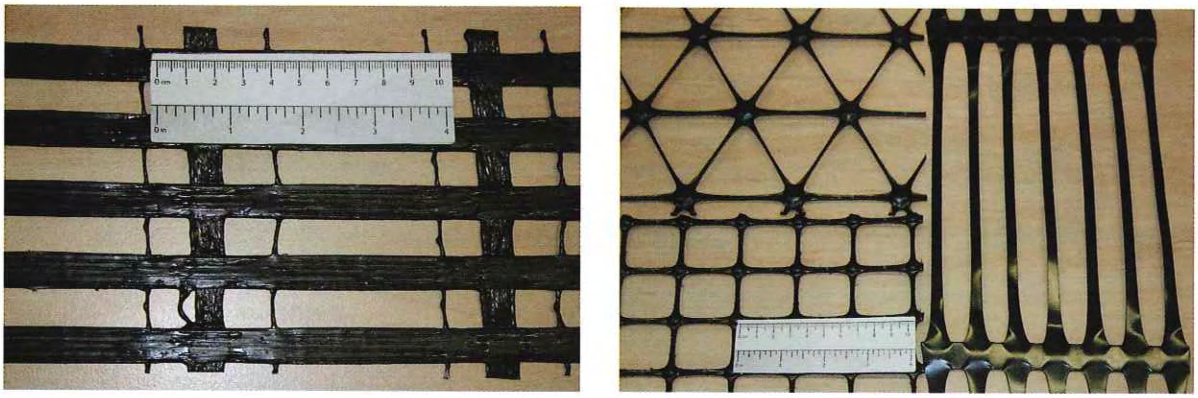
Some distinctions that are worth noting are:

- Polyester (PET) tends to be more expensive than some of the common alternatives, especially polypropylene, but may have a higher stiffness modulus and better creep characteristics. It is heavier than water and therefore will sink in water; hence, it allows construction in water. It is not as sensitive to ultraviolet (UV) light as, say polypropylene, but will deteriorate if left in direct sunlight. Carbon black may be added to improve UV stability.
- Polypropylene (PP) is relatively cheap and can be expected to perform well both wet and dry. However, the specific gravity is less than unity and hence the geotextile may float on water, which could lead to construction difficulties if it has to be placed in water. Polyethylene geotextiles are usually black to reduce the effects of UV light; nevertheless, the PP geotextiles are very sensitive to UV light, and they may lose their strength significantly when exposed to direct sunlight.
- Polyamide (PA) (nylon) may absorb significant amount of water if soaked. When soaked, the strength may be reduced. Carbon black is commonly added to improve UV stability.
- Polyethylene (PE), like PP, is less dense than water and may give rise to construction difficulties if placed in water. The properties of PE may vary significantly and depend on the molecular weight, crystallinity (density), and degree of stabilization. Various high-density polyethylene (HDPE) products are made by the use of special additives (for example, geotextiles or geogrids for reinforcement applications).

6.2.4 Geogrids

Geogrids are matrix-like materials made up mostly of longitudinal and transverse ribs, with apertures typically of size 10 to 200 mm (0.4 to 7.9 in.) (Figure 6.4a and b). However, there are geogrids that have been manufactured recently with triangular apertures intended for use under three-dimensional (3D) loading conditions (for example, the upper left geogrid in Figure 6.4b). The key feature of geogrids is the

Figure 6.4 Examples of geogrids: (a) welded mesh (b) extruded grids



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

aperture size is large enough to allow the strike-through of larger soil particles or rock from one side of the grid to the other.

The major function performed by geogrids is reinforcement. Reinforcement occurs when the soil strike-through within the apertures bears against the transverse ribs, which transmits the load to the longitudinal ribs via the junctions (where the longitudinal and transverse ribs are joined).

Manufacture

There are two main types of geogrid manufacture. These are:

1. *Welded Mesh.* Two sets of strands or tapes at right angles to each other are welded at their intersections. The strands may be welded during the extrusion process, joining the bands while they are still semiliquid, or the weld may be done as a separate operation. Strands may also be made up of composite materials, such as polyester coated with PVC or latex.
2. *Deformed* (or *Extruded) Grids.* Heavy gauge sheets of polyethylene or polypropylene, with holes punched at regular intervals, are drawn either uniaxially or biaxially to produce apertures that are approximately rectangular or square. A schematic of the manufacture of a typical extruded uniaxial geogrid is shown in Figure 6.5. The strength of these types of geogrids is a result of strain hardening in the drawing process.

The first group are characterized by having low flexural rigidity values and are commonly referred to as *flexible* geogrids. The second group are characterized by their homogeneous nature and higher flexural rigidity values. These are commonly referred to as *stiff* geogrids.

6.2.5 Geonets

Geonets are grid-like materials (see Figure 6.6) that are similar to geogrids but are treated separately as they are designed specifically for their in-plane drainage capability, rather than reinforcement. Geonets are always used with a geotextile,

Schematic of a typical extruded uniaxial geogrid manufacture

Figure 6.5

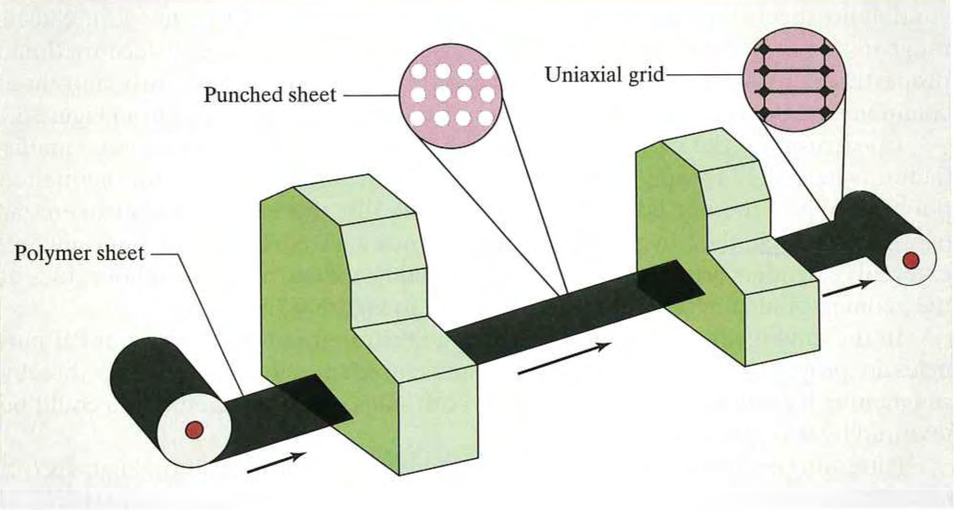


Figure 6.6 Two different typical geonet products: bi-planar (left), tri-planar (right)

(a) (b)

(Courtesy of Geofabrics Australasia Pty. Ltd.)

geomembrane, or other material on both sides to prevent soil intrusion into the apertures, which would block or reduce the in-plane drainage. Almost all geonets are made of polyethylene (PE) resin and are extruded grid-like structures. They are laminated with other materials, either in the manufacturing process, or during laying, to prevent soil intrusion.

6.2.6 Geomembranes

Geomembranes are very-low-permeability synthetic membranes that are used to control fluid or gas migration. Most geomembranes are made from relatively thin, continuous polymeric sheets, but they can also be made from impregnated geotextiles (e.g., impregnation of geotextile with asphalt cement).

Manufacture

Geomembranes are commonly manufactured by the extrusion of polymers into thin continuous sheets that generally vary from 1.8–4.6 m (5.91–15.1 ft) in width. Sheets range in thickness from 0.75 to 3.0 mm (0.03–0.12 in.). To improve surface frictional properties, textured geomembranes have come into the market recently, and three common methods used for their manufacture are illustrated schematically in Figure 6.7.

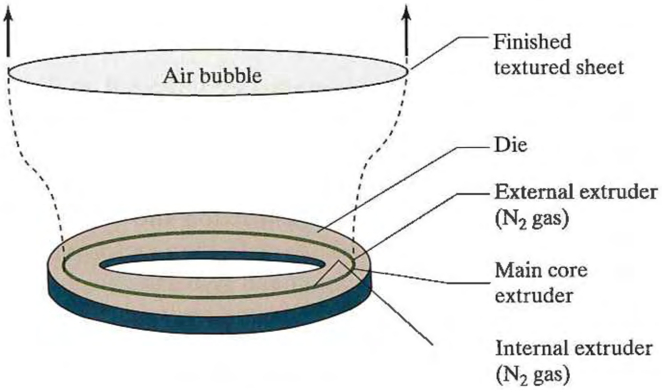
Coextrusion is the most common method of textured PE geomembrane manufacture where the blowing agent, typically nitrogen gas, is injected into the melted polymer. When the extruded polymer exits the die, the bubbles of nitrogen gas expand and burst, thereby producing a roughened surface. Two small (internal and external) extruders are used, adjacent to the main core extruder, when both sides of the geomembrane are to be textured as shown in Figure 6.7a.

In the impingement method of texturing illustrated in Figure 6.7b, hot PE particles are projected on to the previously manufactured smooth geomembrane, thereby roughening its surface. Either one side or both sides of the geomembrane could be textured by this spraying process.

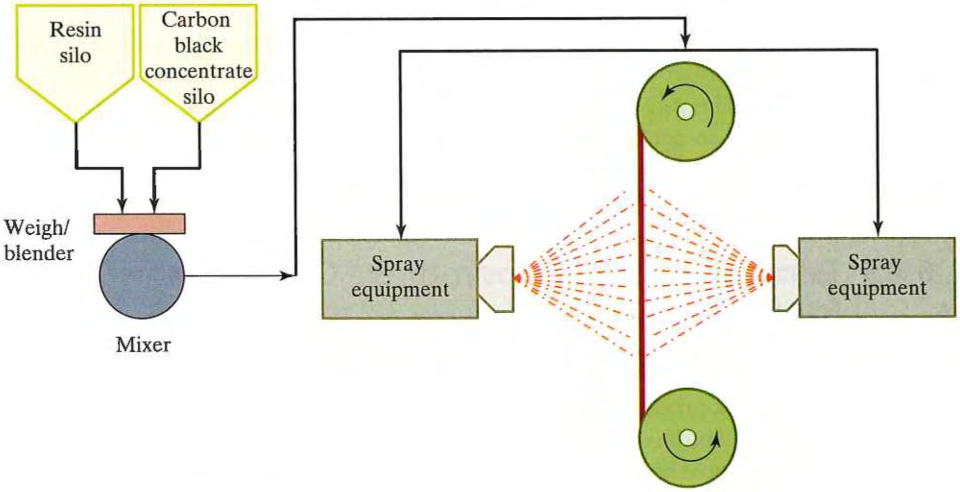
Patterning or structuring is the other method shown schematically in Figure 6.7(c) in which the smooth membrane produced by extrusion is immediately passed through

Methods used to manufacture textured surfaces on HDPE, LLDPE, and flexible PP (fPP) geomembranes

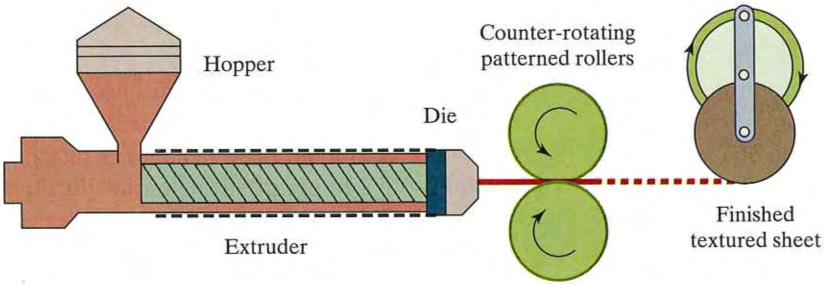
Figure 6.7



(a) Coextrusion with nitrogen gas



(b) Imprigement of hot polyethylene particles



(c) Patterning the surface

(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

two counterrotating inverse patterned rollers, as it exits the lip of the die while it is still hot. Variety of single or multiple raised patterned rough surfaces on the geomembrane sheet could be obtained from this process.

6.2.7 Geocells

Geocells are three-dimensional geosynthetics intended to provide physical confinement to the soil (see Figure 6.8). They are typically made of 100 mm wide and 1.2 mm thick HDPE strips welded ultrasonically at 300 mm intervals. The strips could be planar or otherwise contain punched holes to enhance interaction with the soil. They are usually transported to the construction site in collapsed form and then expanded in place and filled with sand or gravel, thereby forming a geocell-reinforced soil system. Filling of the soil is often continued above the height of the geocell, and the fill material is compacted, typically using a hand-operated vibratory plate compactor.

Geocells are commonly used for construction on soft soils (for example, road construction on weak soils), erosion control on slopes, and channel protection.

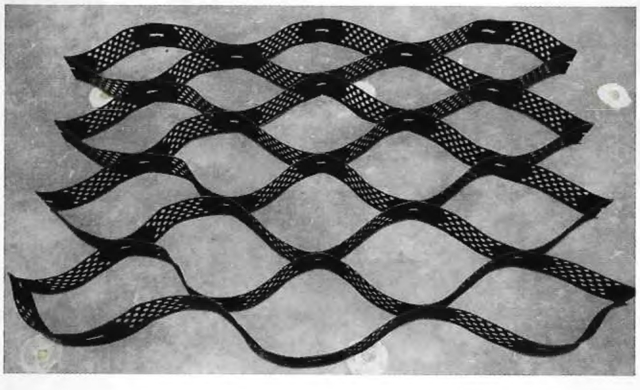
6.2.8 Geomats

Geomats are three-dimensional, net-like open structured geosynthetics, usually made of monofilament polymers, such as HDPE, and heat bonded at the contact points (see Figure 6.9). The open structure allows vegetation across the geomat, thereby providing protection against raindrops and runoff. Hence, geomats are commonly used for revegetation and erosion control solutions in civil engineering projects, such as in cut, as well as fill, slopes.

6.2.9 Geocomposites—Geosynthetic Clay Liners (GCLs), Prefabricated Vertical Drains (PVDs), Geopipes, Geofoams, and Others

Geocomposites are products made of two or more materials stuck together such as geotextile-geonet composite where the geotextile performs the filtration function while the geonet does the drainage function (see Figure 6.10). There are several types

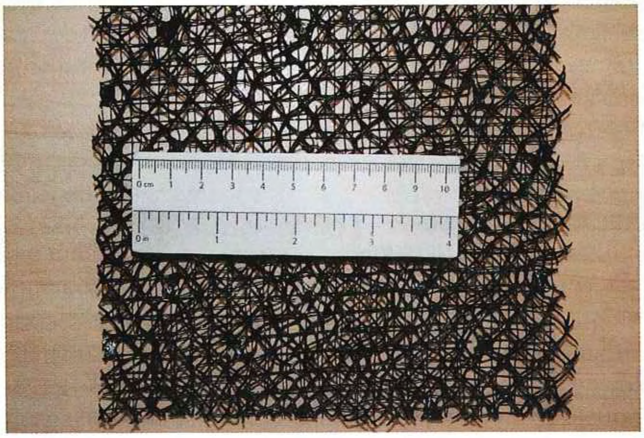
Figure 6.8 Photo of a typical geocell



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Photo of a typical geonet

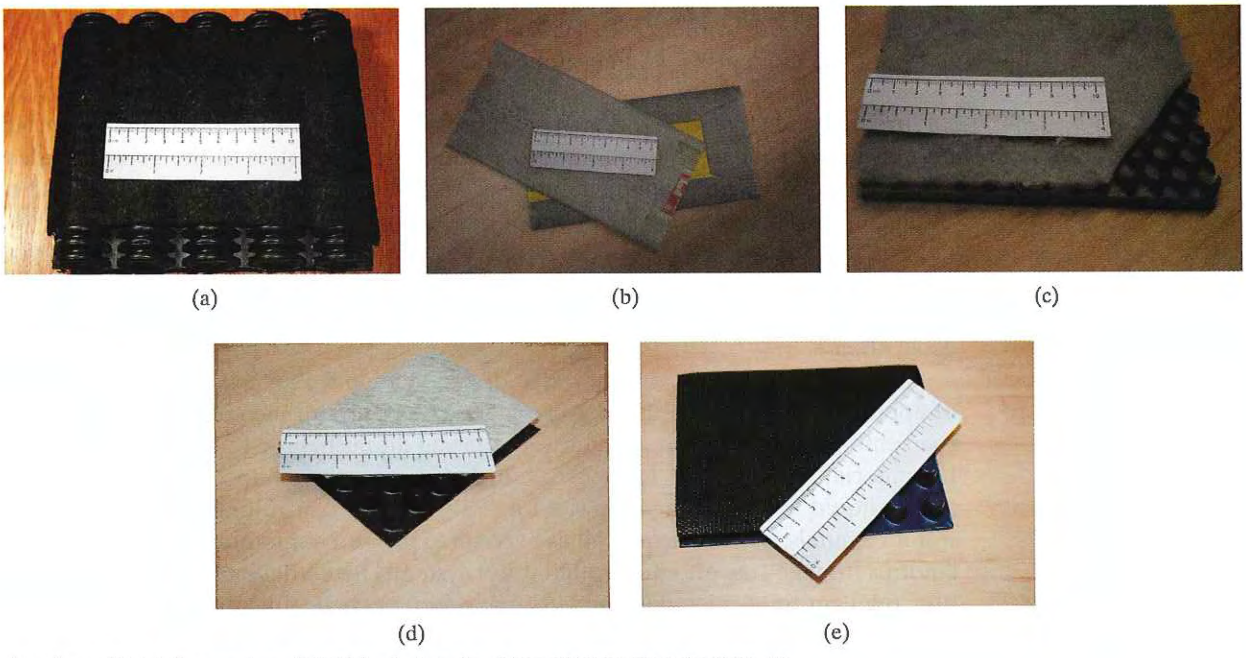
Figure 6.9



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Examples of geocomposites: (a) geotextile-geopipe geocomposite consisting of nonwoven geotextile filter and perforated PP pipe for drainage (b) prefabricated vertical drain (PVD) consisting of nonwoven geotextile filter and corrugated drainage core (c) geomembrane—geotextile—geonet—geotextile geocomposite for liquid containment with drainage and filtration (d) dimpled PP drainage layer with nonwoven geotextile filter (e) dimpled PP drainage layer with woven geotextile filter

Figure 6.10

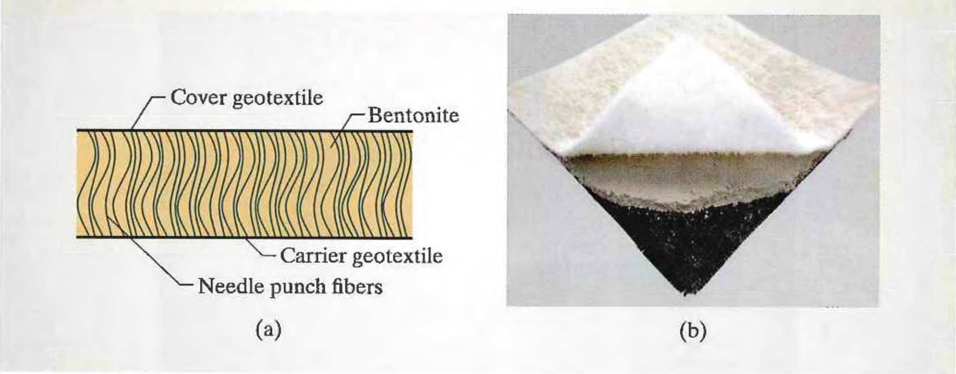


(a) (b) (c)

(d) (e)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 6.11 (a) Schematic of a GCL (b) Sample of a typical GCL, showing its constituents



(a)

(b)

(Courtesy of Geofabrics Australasia Pty. Ltd.)

of geocomposites produced for various civil engineering applications, and the common ones are geosynthetic clay liners (GCLs), prefabricated vertical drains (PVDs), geonet-geotextile composite, geopipes, and geofoams.

GCLs are commonly manufactured with two layers of geotextiles sandwiching sodium bentonite, needle punched together using high-strength fibers to provide effective sealing of liquids against migration (see Figure 6.11). They are commonly used in ponds for water containment, as linings for water channels to minimize leakage loss, and as liquid barriers in waste disposal facilities, such as municipal waste landfills for preventing or controlling contaminant migration.

6.3 Properties and Testing of Geosynthetics

To facilitate the engineering design or the selection of the appropriate geosynthetic for a particular application, you need to know the properties of the geosynthetics. The use of geosynthetics in civil engineering projects mean they are exposed to a wider range of stress, soil, water, environmental, and other conditions. Although it is not possible to test their properties under all such conditions, a number of tests have been developed over the years for determining the properties of geosynthetics under standardized conditions to facilitate the design and the selection of an appropriate geosynthetic.

Geosynthetics cover a wide range of products, and they are used for a variety of applications. For example, they can work as basal reinforcement for constructing road embankments on soft ground; as filters and drains behind retaining walls for the fill material; as separators for constructing roads on poor roadbed soils (subgrades); and as liquid barrier-lining material for preventing or controlling leakage in ponds, water channels, and canals, as well as barriers to prevent contaminated leachate from landfills from getting into the ground water system. Hence, depending on the application, relevant properties, determined from appropriate testing methods are used for their design or for selecting the most suitable geosynthetic for that use.

Sampling, testing protocols, and reporting of test results are performed in accordance to the relevant standard such as the American Standard of Testing Materials (ASTM), Australian Standard (AS), or British Standard (BS). The tests that are commonly carried out are briefly summarized in Table 6.4, and they fall into the following categories:

- Tensile properties and testing
- Geosynthetic–fill (soil) interaction properties and testing (i.e., mainly for reinforcement function). This typically involves the testing of geosynthetic–soil interface shear and geosynthetic pullout behavior characteristics.
- Hydraulic- or flow-related properties and testing (i.e., mainly for filtration and drainage functions)
- Endurance, integrity, durability, and long-term performance-related properties (including puncture, tear, and other installation damage properties) and testing

Common test methods

Table 6.4

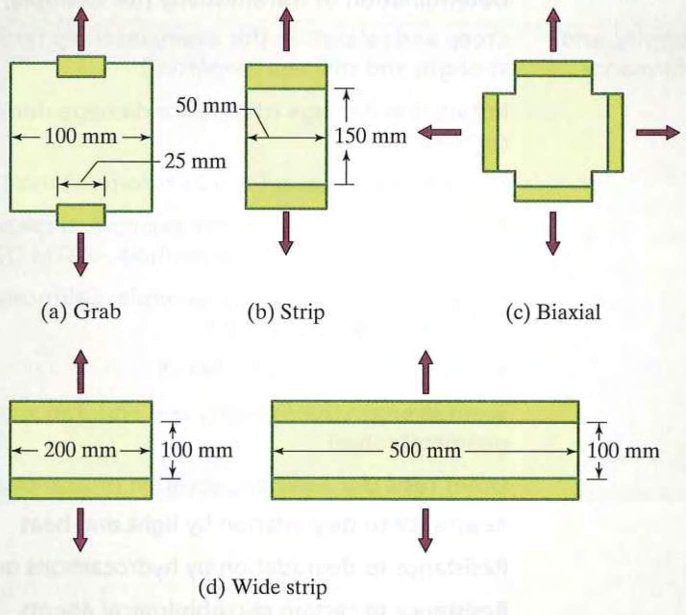
Category	Description
Tensile properties	Determination of tensile properties, such as tensile strength and stiffness; for example, wide-strip tensile test, grab tensile test, biaxial tensile test, (narrow) strip tensile test (see Figure 6.12)
Geosynthetic–fill interaction properties	Geosynthetic–fill interface direct shear test Geosynthetic pullout test
Hydraulic- or flow-related properties	Determination of pore-size distribution—dry-sieving method; wet-sieving method Determination of permittivity Determination of transmissivity (for example, radial flow method)
Endurance, integrity, and long-term performance	Creep and relaxation (for example, creep tensile testing for long-term tensile strength and stiffness properties) Installation damage (that is, for damage during geosynthetic placement and construction) • Drop cone impact (or puncture-resistance) test • Tearing strength test (for example, trapezoidal tear method—ASTM D4533, ISO 13434; tongue tear method—ASTM D751) • Burst strength test (for example, California Bearing Ratio (CBR) plunger method—ASTM D6241) • Puncture test—ASTM D4833 Seam strength (for integrity and endurance testing purposes, typically for geomembranes) Other tests (for example, abrasion resistance test, fatigue test)
Durability	Resistance to degradation by light and heat Resistance to degradation by hydrocarbons or chemical reagents Resistance to certain microbiological agents

6.3.1 Tensile Properties and Testing

For geosynthetics, being generally planar products, tensile strength and stiffness are important characteristics needed for selecting the most suitable geosynthetic for use in a civil engineering structure. Consequently, different tensile testing methods have been developed over the years for determining their tensile stress—strain or load—deformation behavior, tensile strength, and stiffness characteristics, illustrated in Figure 6.12. The testing protocols for each of them are standardized by organisations like ASTM, AS, and BS. Only brief outlines of the common test methods are discussed here, and for details and testing protocols, one should refer to the respective standard.

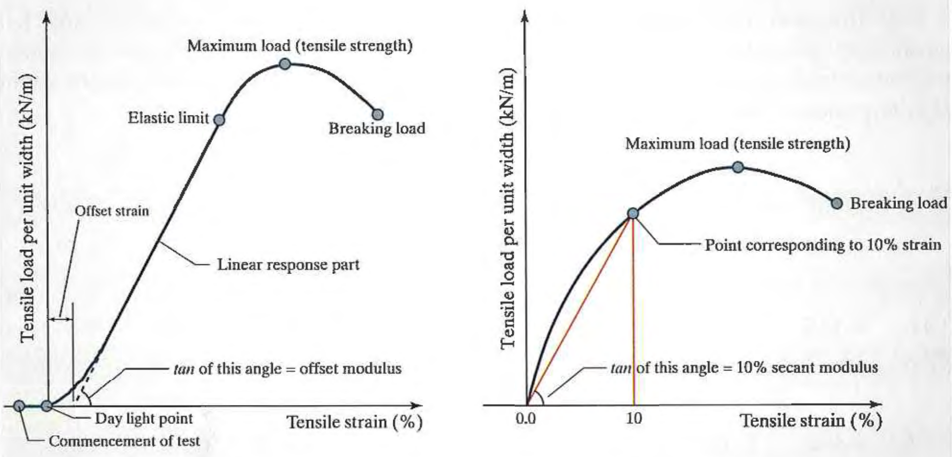
Displacement control uniaxial tensile testing (i.e., test configurations indicated in Figure 6.12a, b, and d) is usually performed by applying a certain rate of tensile elongation to the geosynthetic specimen of standard dimension clamped (without slippage at the clamps and geosynthetic undamaged) at two ends and the corresponding tensile resistance (i.e., tensile load) is measured using a proving ring or load cell during the test. This is usually achieved by keeping the clamp of the geosynthetic specimen at one end stationary while the clamp at the other end is moved at the specified rate. This displacement in the direction of the arrows, illustrated in Figures 6.12a, b, and d, induces tensile load and strain in the geosynthetic specimen. The tensile strain in the geosynthetic is often measured using a strain gauge installed in the mid-region of the specimen. In the absence of such direct strain measurement, although less accurate, the average strain determined from the overall displacement of the specimen measured by monitoring the displacement of the nonstationary

Figure 6.12 Different tensile test methods for obtaining stress–strain characteristics



Tensile load-strain responses of geosynthetics and definition of terms

Figure 6.13



(a) Linear behavior

(b) Nonlinear behavior

(Based on Myles, B. and Carswell, I.G. (1986). Tensile testing of geotextiles. Proceedings of the 3rd International Conference on Geotextiles, Vienna, Austria, pp. 713-718. and Shukla, S.K. and Yin, J-H. (2006). Fundamentals of geosynthetic engineering. Taylor and Francis Publishers, U.K.)

clamp could be used as an alternative. The tensile load versus displacement or strain response obtained from the test is plotted. Depending on the type of geosynthetic tested, the tensile load–strain response could be as shown in Figure 6.13a or b. Also illustrated in this figure are the terms used for defining the tensile strength and common types of stiffness moduli used for characterizing the tensile properties of geosynthetics.

Common terms and their definitions are:

- Tensile strength—maximum tensile stress (i.e., maximum tensile load per meter width. Note: although stress is commonly defined as load per unit cross-sectional area, thickness of the geosynthetic is not included, and it is customary to refer to load per unit width as the stress in geosynthetic).
- Strain at failure (i.e., geosynthetic strain at maximum load).
- Toughness—work done per unit volume before failure, usually taken as the area under the stress-strain curve.
- Modulus (of elasticity) or stiffness is generally defined as the slope of the initial linear portion of the stress-strain curve. For geosynthetics, however, there are different moduli values used for characterizing their tensile behavior:
  - Initial tangent modulus—used for geosynthetics that have initial linear stress-strain response.
  - Offset tangent modulus—used when initial slope is very low (see Figure 6.13a).
  - Secant modulus—slope of the line connecting the origin to the point corresponding to a specific strain (for example, 10% strain) on the stress-strain curve (see Figure 6.13b). Depending on the strain level in the geosynthetic during its intended function in the field application, an appropriate secant modulus could be used for the design. For example, 5 percent secant modulus is appropriate if the estimated strain in the geosynthetic under working stress condition is 5 percent.

EXAMPLE 6.1

A wide strip tensile test was conducted on a sample of high strength geotextile and the collected tensile load–elongation data are tabulated below. The width and gauge length of the specimen were measured to be 200 mm and 102 mm respectively. Determine the tensile strength, strain at failure and modulus of this geosynthetic.

Elongation (mm)		0.000	0.2907	0.755	1.265	1.760	2.234	2.703	3.172
Tensile load (kN)		0.000	0.1556	0.389	1.389	3.300	5.322	7.078	8.578
Elongation	3.647	4.116	4.605	5.110	5.569	6.110	6.589	7.048	7.512
Tensile load	10.07	11.54	13.17	14.94	16.64	18.72	20.57	22.38	24.20
Elongation	7.997	8.466	8.925	9.410	9.869	10.34	10.82	11.28	11.77
Tensile load	26.04	27.92	29.72	31.62	33.38	35.08	36.80	38.34	39.84
Elongation	12.24	12.71	13.18	13.64	14.11	14.60	15.08	15.53	15.99
Tensile load	41.06	41.96	42.04	41.26	38.80	38.16	36.80	36.14	35.00

Solution:

The strain in the geotextile could be calculated using the relationship,

$$\text{tensile strain} = \frac{\text{Elongation}}{\textit{Gauge length}} \times 100\ \%$$

$$= (\text{Elongation in mm}) \times \frac{100}{102}\ \%$$

The tensile force per m width = (tensile load)/(specimen width in m)
= (tensile load)/0.2 kN/m

Using these relationships for the given tensile load–elongation data, the tensile force per meter versus strain could be calculated.

For example, considering the data point: elongation = 3.172 mm, tensile load = 8.578 kN;

$$\text{Tensile strain} = 3.172 \times \frac{100}{102} = 3.11\%$$

$$\text{Tensile stress} = \text{tensile load per meter} = \frac{8.578}{0.2} = 42.89\ \text{kN/m}$$

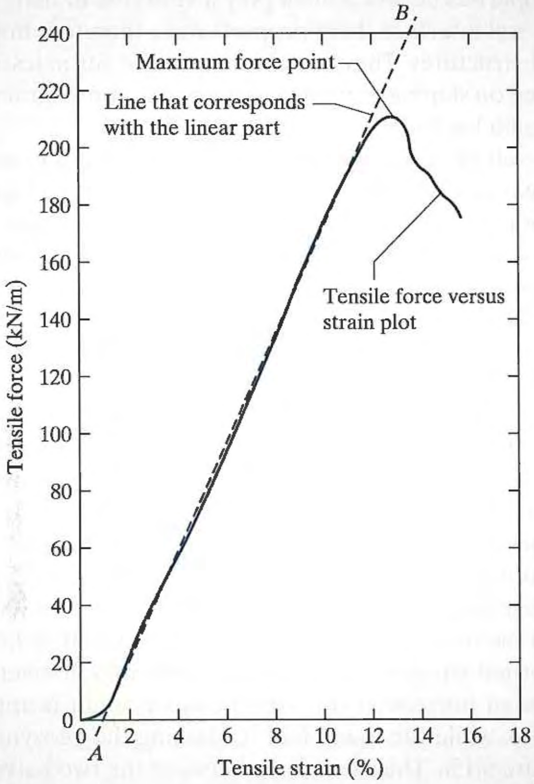
The tensile force per meter versus strain behavior of the geotextile could be plotted as shown in Figure 6.14 for determining the strength and stiffness parameters.

From the plotted results:

Tensile strength of the geotextile = tensile force corresponding to the maximum point
= 210.2 kN/m ≃ 210 kN/m

Tensile force versus strain behavior of geotextile from wide strip tensile test

Figure 6.14



Tensile strain at failure = tensile strain corresponding to the maximum point
= 12.9% = 0.129

For determining the modulus, the line corresponding to the linear part of the tensile force versus strain behavior is identified as shown, and the slope of this line is determined.

$$\text{Offset tangent modulus of the geotextile} = \text{slope of line AB} = \frac{240 - 0}{(13.6 - 0.95)\ \%}$$
$$= 1897 \text{ kN/m} \simeq 1900 \text{ kN/m}$$

6.3.2 Fill-Soil Interface Shear Properties and Testing

One of the main uses of geosynthetics in civil engineering is to perform a reinforcement function in soil structures (for example, reinforced soil walls, steep slopes, roads on soft ground). Soil is weak in tension but capable of supporting compression. Therefore, geosynthetics (either geogrids or geotextiles) are included while constructing the soil structure to support the tension. For the reinforcement to be effective, the tensile stresses have to be transferred from the soil (or fill material) to the geosynthetic reinforcement without any slippage at the geosynthetic–soil interface. This stress transfer happens through the interaction between the geosynthetic and soil, and the interface shear properties between them play a vital role in that.

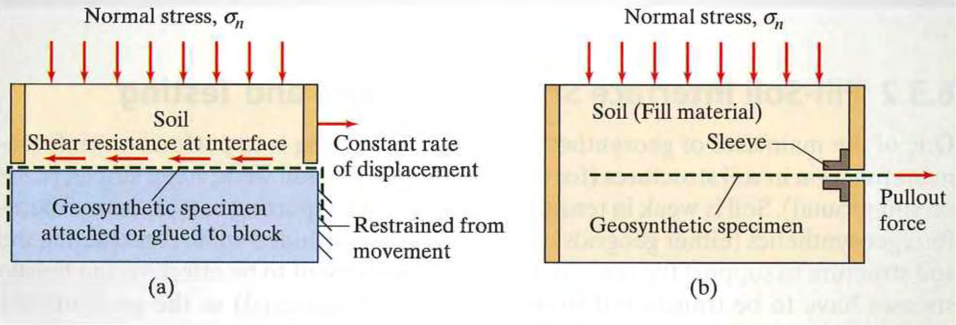
Geosynthetic–soil interface shear properties are therefore important for designing reinforced soil structures. They are also important for assessing the stability of geosynthetics placed on sloping ground (for example, geomembrane and GCL placed on the side slopes of a landfill).

Geosynthetic–soil interface shear properties are generally assessed through an interface direct shear test, and it is illustrated schematically in Figure 6.15a. Geosynthetic anchorage or pullout capacity is also an important consideration for designing reinforced soil structures (for example, a geosynthetic-reinforced soil wall), and it is assessed by a pullout test illustrated in Figure 6.15b.

The interface direct shear test is generally performed in a modified direct shear apparatus such as that shown in Figure 6.15a. The lower half of the shear box contains a metal block with the geosynthetic attached or glued onto it. The upper half of the shear box is used to prepare the soil material to the required consistency (for example, moisture and density) as it is used in the field. The typical size of the interface direct shear apparatus for geotextiles is 300 mm × 300 mm (12 in. × 12 in.). However, depending on the particle size distribution (PSD) of the soil and the characteristics of the geosynthetic, a much larger size shear box may be preferred. For example, for a granular fill soil with maximum particle size of 25 mm (1.0 in.) and a geogrid with approximately 200 mm × 15 mm (8.0 in. × 0.6 in.) aperture size, much larger shear box in the order of 1000 mm × 500 mm (3.28 ft × 1.64 ft) is preferable.

A constant normal stress is applied to the geosynthetic-over-soil combination, and a constant rate of horizontal (or shear) displacement is applied to the upper half of the shear box while the lower half containing the geosynthetic is restrained as indicated in Figure 6.15a. The shear force between the two halves (i.e., shear resistance at the geosynthetic–soil interface) and the shear displacement are monitored during the test. Shear force is typically measured using a horizontally aligned load cell attached to the lower half (i.e., the horizontal force needed to restrain the lower half from movement) and a dial gauge attached to the upper half to monitor its horizontal displacement. The shear stress (= shear force/area of interface) is plotted against the shear displacement, and the maximum shear stress, which indicates the peak interface shear strength, is determined. This procedure is repeated for a few different normal

Figure 6.15 Schematic of (a) interface direct shear apparatus (b) pullout testing equipment



stresses, and the interface shear strength parameters are inferred by plotting the Mohr-Coulomb type shear strength envelope as shown in Figure 6.16.

The slope angle and the intercept of the Mohr-Coulomb type failure (or shear strength) envelope is referred as the peak interface friction angle, δ_p (or $\phi_{int\text{-}peak}$) and peak adhesion, c_{ap} (or $c_{int\text{-}peak}$). In addition to the consideration of the maximum shear stress points (Figure 6.16a), the residual values of shear-stress–shear displacement responses are also considered in a similar fashion, and the corresponding slope angle δ_r (or $\phi_{int\text{-}res}$) and intercept residual adhesion, c_{ar} (or $c_{int\text{-}res}$) are also determined for use in certain design situations. Typically, the interface shear strength (τ_f) is given by:

$$\tau_f = c_{ap} + \sigma \tan \delta_p \tag{6.1}$$

where

σ = normal stress on the geosynthetic

c_{ap} = peak adhesion, and

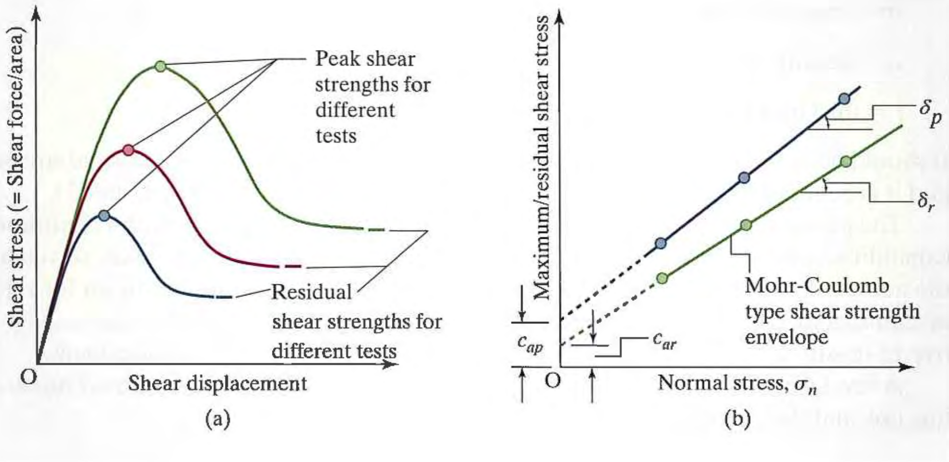
δ_p = peak interface friction angle.

The geosynthetic pullout test is conducted in a box-like apparatus with a slit on one side as shown in Figure 6.15b. The soil is compacted in the box to the required consistency (for example, moisture and density) as it is used in the field, with the protruding geosynthetic embedded as shown. The protruding part of the geosynthetic is clamped, using an appropriate anchoring system to facilitate the pullout. Depending on the size of the pullout box, a hydraulic jack may be used to apply the pullout force or displacement.

In a typical pullout test, a constant normal stress (σ_n) is applied to the geosynthetic–soil system, and a constant rate of pullout displacement is applied to the geosynthetic. The pullout force (or load) and the displacement of the geosynthetic are monitored during the test. The pullout load displacement results are plotted to

Illustration of typical interface direct shear test results: (a) shear stress–displacement response (b) Mohr-Coulomb type shear strength envelope

Figure 6.16



determine the maximum pullout force, which indicates the pullout capacity of the geosynthetic. The pullout capacity is dependent on the geosynthetic–soil interaction and not just twice the force corresponding to direct shear strength (i.e., shearing on both upper and lower surfaces). This is because the geosynthetic reinforcement deforms in pullout, whereas it does not in direct shear.

There are apertures and ribs in a typical geogrid. Hence during the pullout of a geogrid, there is soil–soil shear in the aperture, soil–geogrid planar surface shear, and bearing failure of the soil on the vertical areas of geogrid-rib. Equations have been proposed in the literature to estimate the pullout capacity of a geogrid considering all these factors (see Jewell et al. 1984).

6.3.3 Hydraulic- or Flow-Related Properties and Testing

As is discussed later in Section 6.4, filtration is one of the major functions a geosynthetic is used to perform in a civil engineering structure. In the filtration function, the geosynthetic is used for preventing soil and other solid particles from passing through while allowing water (or fluid) to pass. Geotextiles, particularly nonwoven geotextiles, are commonly used as filters in civil engineering applications.

The openings in between the fibers are generally referred as pores and voids in the geotextile, and they play an important role in the performance of the filtration function. It is imperative to examine them closely. There are certain terms that are used for characterizing the pores and voids in a geotextile, and they are examined below.

The first term considered in this regard is the *porosity*, which is a volumetric measure of the open space referred to as voids in the geotextile relative to unit volume of geotextile. It is defined as:

$$Porosity\ of\ a\ geotexile\ (n) = \frac{Volume\ of\ void}{Total\ volume} \tag{6.2}$$

Porosity is not measured directly but calculated from other properties of the geotextile, typically using the equation:

$$n = 1 - \frac{m}{\rho t} \tag{6.3}$$

where

m = mass per unit area (g/m^2)

ρ = density (g/m^3)

t = thickness (m)

It should be noted that the thickness of a geotextile is dependent on the normal stress, and it is generally measured under a standard normal stress of 2 kPa (42 lb/ft^2).

The pore size of a geotextile is also important for assessing its flow and filtration capabilities, and it could be measured by sieving controlled-size glass beads through the geotextile. Published values of pore size, thickness, and so forth are often for the as-manufactured geotextile. However, it is imperative to note that pore size is sensitive to in situ stress conditions on the geotextile and gradations of adjacent soil.

A few terms are used to characterize the pore size of a geotextile, and their meaning, use, and determination methods are highlighted below.

Percent Open Area (POA)

Percent open area (POA) is a property that characterizes the pores (or openings) in a geotextile, used only for monofilament woven geotextiles. POA is the ratio of the total open area (i.e., the void areas between adjacent yarns) to the total specimen area, that is,

$$POA = \frac{Total\ open\ area\ \text{in}\ the\ specimen}{Total\ area\ of\ specimen} \quad \textbf{(6.4)}$$

It is commonly measured by projecting a light through the geotextile onto a screen, mapping the magnified open spaces on the screen, and measuring the areas of open spaces by a planimeter. POA is an important property for assessing the water flow and soil retention capabilities of monofilament woven geotextiles when such a geotextile is considered for use as a filter (for example, as a silt fence on slopes to control erosion).

Apparent Opening Size (AOS) or O_{95}

Apparent opening size (AOS) or O_{95} is the approximate diameter of the largest particle that would effectively pass through the geotextile. At least 95 percent of the openings of the geotextile apparently have that diameter or are smaller, as measured by a dry sieve test (ASTM D4751).

AOS is generally determined by sieving (i.e., either dry or wet sieving) glass beads or clean quartz sand of known particle size (i.e., particle diameter) and measuring the percent of the weight of glass beads that pass through the geotextile specimen. Successively different one-diameter glass beads are sieved, and the percent passing is determined for each diameter of glass beads. The diameter of glass beads (in mm) corresponding to 5 percent passing through the specimen is determined, and it is referred to as the AOS or O_{95} of the geotextile. Unlike POA discussed earlier, the AOS (or EOS or O_{95}) is used for all types of geotextiles.

The glass bead diameter corresponding to 5 percent by weight passing through defines the O_{95} size of the geotextiles openings in millimeters. O_{95} size defines only one particular opening size of the geotextile, and it does not give the whole pore size distribution (see Figure 6.17 below for an illustration of pore size distribution of two geotextiles A and B and the inference of O_{95} size of each of them). For example, the AOS of geotextile A shown in Figure 6.17 is 0.2 mm (0.008 in.). It is worth noting that the AOS, Equivalent Opening Size (EOS), and O_{95} refer to the same specific pore size.

Cross-Plane Permeability and Permittivity of Geosynthetics

When a geotextile is used as a filter, it is expected to be sufficiently permeable to facilitate water (or fluid) flow across it, as illustrated in Figure 6.18. Therefore, in addition to its AOS, it is important know the across-the-plane permeability (k_g) of the geotextile. However, the thickness (t) of the geotextile filter could change, depending on the normal stress acting on it which in turn could influence the cross-plane permeability. Hence, the term permittivity (ψ), is defined by the equation:

$$\psi = \frac{k_g}{t} \quad \textbf{(6.5)}$$

Figure 6.17 Pore size distribution curves of two geotextiles

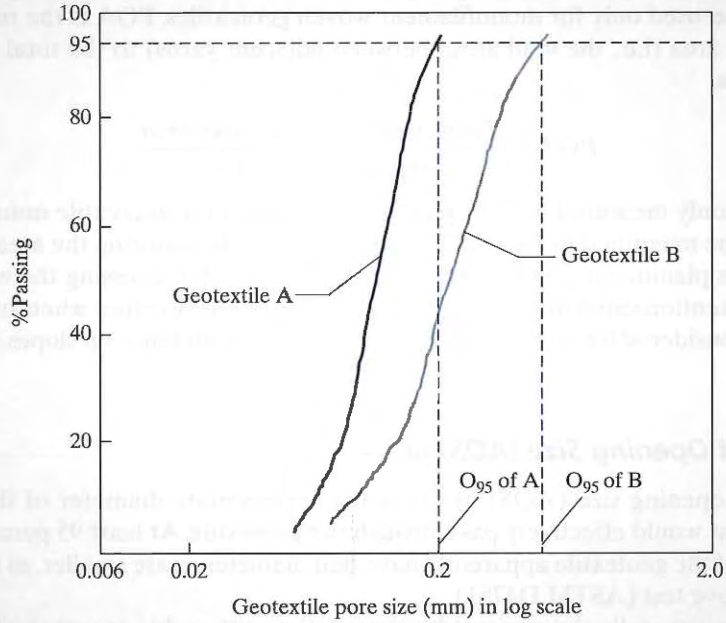
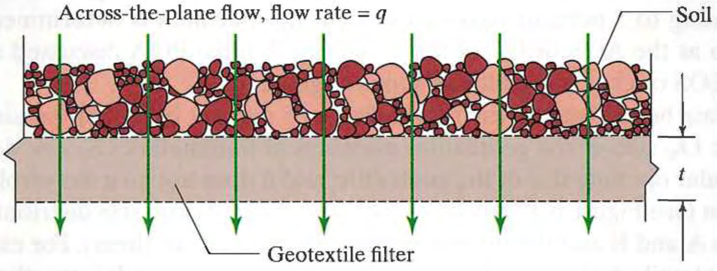


Figure 6.18 Schematic of a geotextile filter



where

ψ = permittivity of geotexile

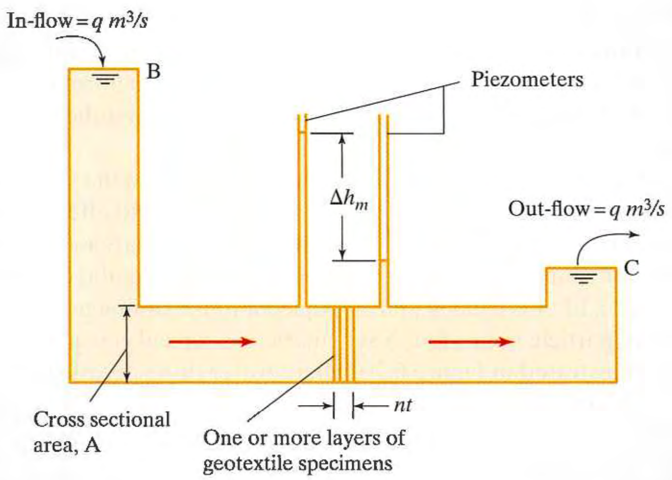
kg = across-the-plane permeability of the geotextile

t = thickness of the geotextile

The permittivity, permeability, and flow rate of a geotextile are often determined using a constant head permeameter such as that illustrated in Figure 6.19. The apparatus is usually transparent, with constant water levels at B and C, the elevation being higher at B compared to that of C. It is capable of supplying and controlling flow across the geotextile specimen(s) with a steady state velocity ranging from 0 to 0.035 m/s (0 to 0.115 ft/s). One or more layers of geotextile specimens are mounted vertically as

Figure 6.19

Schematic of a permeameter used for determining across-the-plane flow properties of geotextile



(Based on Australian Standard AS 3706.9, 2001)

indicated, usually supported by a grid. The method is based on Darcy's law for laminar flow, which is relevant for common groundwater flow. Hence the permeability obtained is generally referred as Darcy coefficient of permeability.

When steady state is reached, the head loss across the geotextile (i.e., Δh_m in m) and the flow rate (i.e., inflow rate = outflow rate = q m^3/s) are measured. The measured head loss (Δh_m) is often corrected by deducting the head loss in the apparatus plus the supporting grid alone (i.e., without the geotextile specimens). If the corrected head loss across the geotextile specimen(s) is Δh (in m), the permittivity is determined using:

$$\psi = \frac{q\,n}{A\,\Delta h} \qquad \textbf{(6.6)}$$

where

ψ = permittivity of geotextile (s^{-1})

q = flow rate at steady state (m^3/s)

n = number of geotextile specimens

A = area of geotextile specimen (m^2)

Δh = corrected head loss across the geotextile (m)

Across-the-plane (Darcy coefficient of) permeability of the geotextile, $k_g = \psi t$, and its common unit is mS^{-1}

Flow rate per square meter of geotextile under a constant water head, $Q = \psi\,\Delta h\,A$ and its unit is m^{3}S^{-1}

Where t is the thickness of a specimen (unit: m), n is the number of geotextile specimens used, and A is the cross-sectional area of the test specimen exposed for the flow (unit: m^2).

In-Plane Permeability and Transmissivity of Geosynthetics

Geonets, geotextiles, and geonet-geotextile composites are the types of geosynthetics commonly used for drainage applications. For a geosynthetic to function as a drainage medium, it should be sufficiently permeable for flow in its plane. The in-plane permeability of the geosynthetic (k_p) for the drainage function is dependent on the thickness of the geosynthetic, which could be affected by the normal stress and other factors. It should be noted that the thickness (t) of the geosynthetic could decrease with increasing normal stress, particularly for geotextiles.

In a typical drainage application, fine solid particles from the soil upstream could migrate with the water flowing into the geosynthetic drain and affect its performance. These solid particles could reduce the thickness and open area of the geosynthetic drain available for transmitting water and in extreme cases could clog the drain altogether. Therefore, a filter is usually placed adjacent to the drainage layer to prevent or reduce such solid particle migration. A schematic of a typical geosynthetic drain–filter arrangement is illustrated in Figure 6.20, where water flows across the filter and then flows along the plane of the geosynthetic drain.

Transmissivity (θ) is the term used for characterizing the geosynthetics for in-plane flow and drainage applications. It is defined as:

$$\theta = k_p\, t \tag{6.7}$$

where θ = transmissivity of geosynthetic (m^2/s

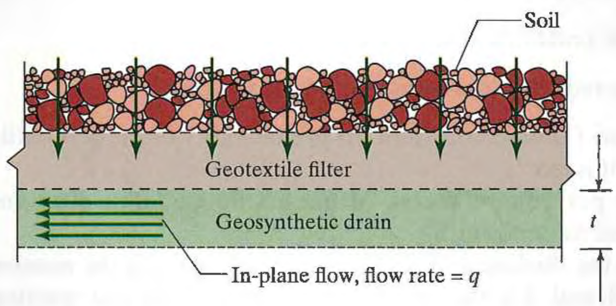
t = thickness of the geosynthetic (m)

k_p = in-plane permeability of the geosynthetic (m/s)

There are different methods available for determining the transmissivity of a geosynthetic, and a typical method used for nonwoven or composite geotextiles is illustrated in Figure 6.21.

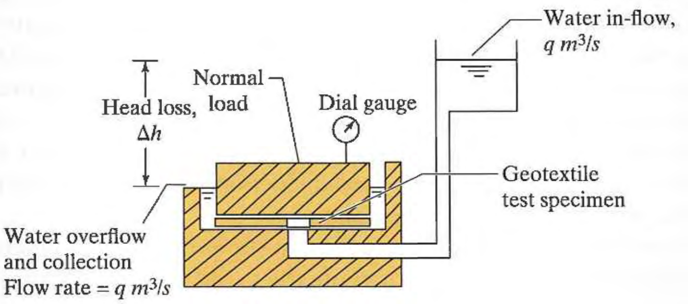
In this method, a circular geotextile test specimen of 150 mm (6 in.) radius with a 25 mm (1 in.) radius hole cut out in the center is typically used in between two metal plates of the same dimensions. A normal load corresponding to 100 kPa (1 ton/ft^2) normal stress on the specimen is applied. The apparatus is designed such that water enters the geotextile specimen at the center annular hole and flows radially outward in the plane of the geotextile as shown in Figure 6.21.

Figure 6.20 Schematic of a geosynthetic drain application



Schematic of transmissivity test of geotextile–radial flow method

Figure 6.21



(Based on Australian Standard AS 3706.10, 2001)

When steady state is reached, the head loss in the geotextile (i.e., Δh in m) and the flow rate (i.e., when inflow rate = outflow rate = q m^3/s) are measured. The transmissivity (θ, in m^2/s) of the geotextile is determined using:

$$\theta = \frac{q}{2\pi\,\Delta h}\ln\left(\frac{r_1}{r_o}\right) \tag{6.8}$$

where

θ = transmissivity of geotextile (m^2/s)

r_l = outer radius of the two plates (mm)

r_o = radius of the water supply inlet (mm)

Δh = head loss in the geotextile at steady state (m)

q = flow rate at steady state (m^3/s)

In-plane permeability of the geotextile is $k_p = \frac{\theta}{t}$, where
t = thickness of the geotextile test specimen (in m).

6.3.4 Endurance, Integrity, Durability and Long-Term Performance-Related Properties and Testing

A geosynthetic is expected to perform adequately throughout its intended design life, and to ensure its endurance, integrity, durability, and long-term performance, several tests have been developed. As seen previously, a wide range of geosynthetics (for example, geotextiles, geonets, composites, etc.) is produced to perform different functions (i.e., reinforcement, filtration, liquid barrier, etc.). For example, geonets are primarily used for drainage applications, geomembranes are used mainly for (liquid) containment, and geogrids are used as reinforcement. Consequently, some of the tests for assessing endurance, integrity, and long-term performance are geosynthetic-type specific, which is highlighted when discussing each of the common tests.

Endurance and Integrity of Geosynthetics

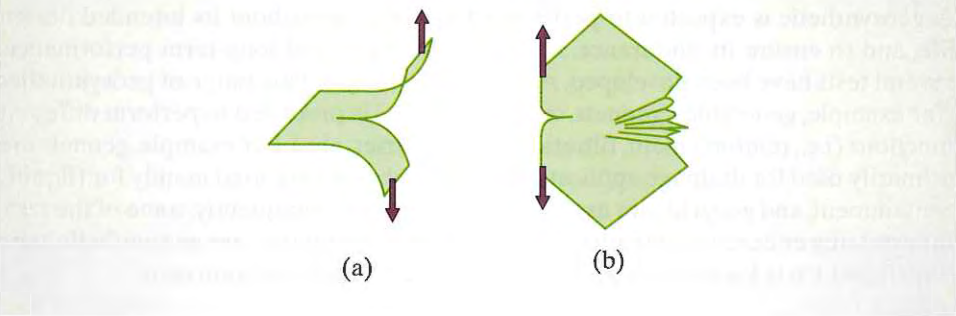
Geosynthetic products and their applications range widely, and several geosynthetic-type specific endurance and integrity tests have been developed to facilitate the selection of geosynthetic and engineering design. Although not all the properties and tests carried out to ascertain endurance, integrity, and long-term performance of geosynthetics are presented, the common tests and their important aspects are discussed in this section. Some of the geosynthetic-type specific tests, particularly for geogrids, geonets, and geomembranes, are discussed in Section 6.4 while considering the functions mechanisms and engineering applications.

The tear propagation test, puncture or burst strength test, and dynamic puncture test are the common integrity tests for geotextiles, and they are discussed below.

Tear Propagation Test: This is to test the ability of a geosynthetic, particularly a nonwoven or woven geotextile, to withstand stresses subsequent to a tear, typically generated during the installation or construction. The objective is to assess whether and when the tear would continue to propagate, and it is done on a geotextile specimen of standard shape and size with a particular size tear initially made in it. There are two common types of tests, namely (1) load applied normal to the plane of geotextile, referred as wing tear or tongue tear test (e.g., BS 3424) and (2) load applied in the plane of geotextile, commonly referred as trapezoidal tear test (e.g., ASTM D4533, AS 3706.3); these are illustrated schematically in Figure 6.22. In these tests, a constant rate of tensile extension (or elongation) is applied, and the tensile force resistance (i.e., tensile load) in the geotextile is measured. The tensile load–extension behavior of the specimen is monitored, and its tear strength is determined from the maximum load.

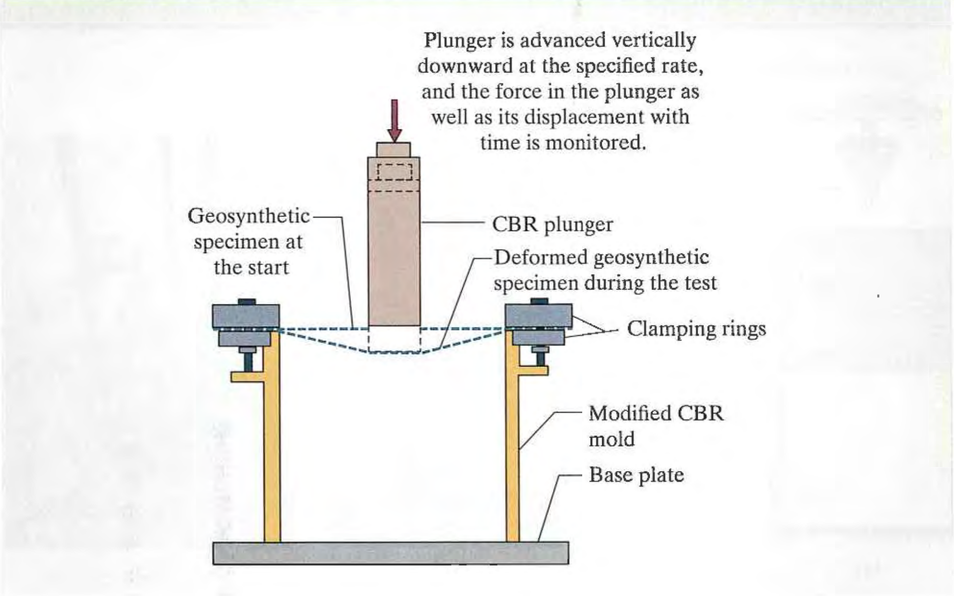
Static Puncture or Burst Strength Test This is to test the ability of a geosynthetic, particularly a nonwoven or woven geotextile, to withstand localized stresses generated by penetrating or puncturing objects, such as aggregates or tree roots, under quasi-static loading conditions. The common burst strength test is the California Bearing Ratio (CBR) plunger method (ASTM D6241, AS 3706.4, ISO12236) that is illustrated in Figure 6.23. In this method, a circular geotextile specimen is clamped securely on a CBR mold (typically 152 mm inner diameter), and the CBR plunger (typically 50 mm (2 in.) diameter and 150 mm (6 in.) long) is advanced vertically downward at

Figure 6.22 Illustrations of tear propagation tests: (a) wing tear, or tongue tear, test (b) trapezoidal tear test



Schematic of CBR plunger burst strength test

Figure 6.23



the specified rate (typically at 50 mm (2 in.) per minute). Force in the plunger is measured, using a load cell or proving ring, and its displacement is monitored with a dial gauge or LVDT. The maximum plunger load is reported as the CBR plunger burst strength of the geotextile. The corresponding geotextile strain is also usually reported.

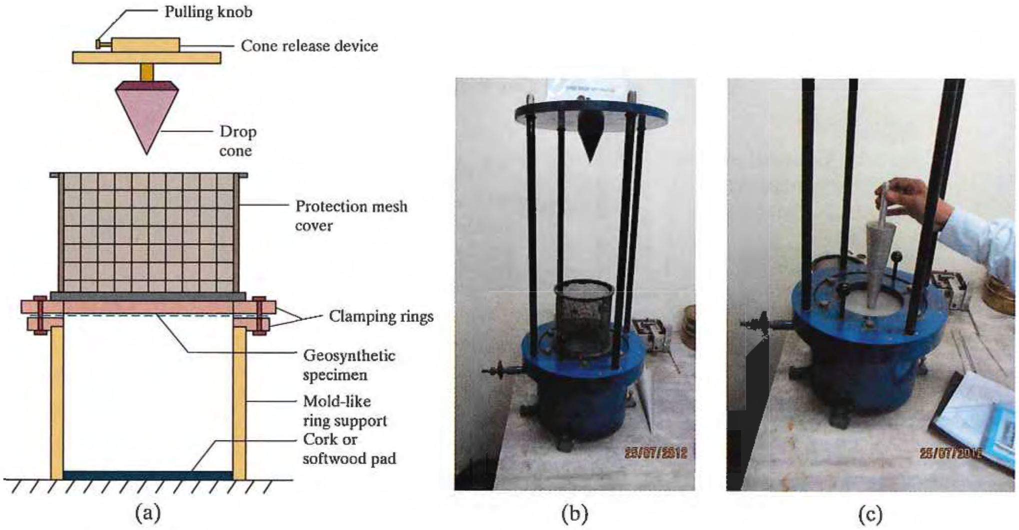
Dynamic Puncture Strength Test This is used to test the ability of a geosynthetic, particularly a nonwoven or woven geotextile, to withstand stresses generated by the sudden impact and penetration of falling objects such as coarse aggregates, tools, and other construction items during the installation process. The drop cone puncture test is the most common of the puncture tests performed. A standard size cone (typically 1 kg mass, 50 mm (2 in.) maximum diameter and 45° apex angle) is dropped from a specified height, and the size of the puncture it makes on the geotextile is measured, using a standard measuring cone. The geotextile specimen of standard size is commonly clamped on a mold-like ring support (or CBR-type mold) before the drop cone is released, as illustrated in Figure 6.24.

Long-Term Performance of Geosynthetics

Long-term performance of a geosynthetic is commonly characterized by two phenomena called creep and relaxation, which are commonly referred to as time-dependent stress-strain behavior characteristics. Creep refers to the increase in strain (or elongation) in the geosynthetic with time under sustained loading. On the other hand, relaxation refers to the decrease in the tensile force in the geosynthetic with time when it is under sustained strain (or elongation).

Creep tensile behavior of a geosynthetic is often studied by applying a constant tensile force, typically by hanging weights or a tensile load application system to a vertically held geosynthetic specimen under constant temperature and humidity (i.e., controlled environmental) conditions and monitoring the elongation or strain

Figure 6.24 Drop cone puncture test: (a) schematic description of the equipment (b) photo of the equipment (c) illustration of puncture measurement



(Courtesy of N. Sivakugan, James Cook University, Australia)

in it with time. This is generally a long duration test that is carried out in steps of increasing tensile force, and the tensile strain is monitored at intervals of time, typically over 10,000 hours at each tensile load step. The results are analyzed by plotting the tensile strain versus log time (S).

Studies have shown that strength as well as stiffness of a geosynthetics is time dependent. Jewell (1985) pointed out that when subjected to a sustained stress in a constant chemical and temperature environment, the time to failure increases with decreasing levels of stress, the cumulative strain to failure decreases with decreasing levels of stress, and polymers which are ductile in the short term become less ductile as the level of sustained stress decreases and time to failure increases. Moreover, susceptibility of a geosynthetic to creep will depend on the method of manufacture, the polymer used (e.g., among commonly used geosynthetic reinforcement products, PE and PP tend to creep more than PET), and damage to the geosynthetic or the aggressive nature of the chemical environment (Jewell, 1985). Therefore, for applications where the geosynthetic is expected to sustain loads for a prolonged period of time and the stability of the structure is dependent on the contribution of the geosynthetic (e.g., geosynthetic reinforced soil walls and steep slopes), the variation of strength and modulus with time is of paramount importance. Hence, appropriate safety factors are allowed to account for the reduction in strength and stiffness of geosynthetic due to creep with time when designing such reinforced soil structures.

Geosynthetics are relatively new products for use in civil engineering construction, and a major limitation arises for designing structures that have prolonged life in the order of 50–100 years because the appropriate creep data is not yet known. However, the effects of an increase in temperature on the stress-strain behavior of geosynthetics have been found to relate to that of time, and it has been used to extrapolate the time and predict the long-term creep behavior.

Durability of Geosynthetics

Sunlight Degradation: If geosynthetics are exposed to sunlight over an extended period of time, degradation can occur due to UV radiation breaking down the polymers from which geosynthetics are made. In most applications geosynthetics are covered with soil and/or rock, so once placed, no UV degradation can occur. However, care must be taken during construction and in the storage of the geosynthetic if it is purchased well in advance of its use. For long-term performance, it is essential that the geosynthetic is sufficiently covered to protect it from UV radiation, both when in storage as well as when in use.

Temperature Degradation: High temperature causes polymer degradation to occur at an accelerated rate. Temperature in itself does not cause the degradation; it accelerates the degradation caused by other mechanisms. High temperatures increase the flexibility of geosynthetics. While this does not degrade the geosynthetic, it alters its properties and may change its serviceability for an intended application.

Oxidation degradation: All polymers react with oxygen causing degradation; however PP and PE are considered to be most susceptible. The degradation caused by oxidation increases as temperature increases.

Other Degradation Processes: Geosynthetics may be subject to a number of other mechanisms that can cause degradation. These may include:

- Hydrolysis degradation, caused by immersion in liquid of either extremely high or extremely low pH.
- Chemical degradation, caused by chemical reagents present in either the ground water or surrounding soil.
- Rodent or termite attack.
- Abrasion caused by the movement of abrasive soil particles over or through the geosynthetic, particularly during placement and construction. This is also generally included in the installation damage when estimating the available strength and other properties for a particular application.

6.4 Functions, Mechanisms and Engineering Applications

Geosynthetics are used to perform one more of the functions of reinforcement, separation, filtration, drainage, containment (i.e., mainly liquid or vapor barrier), and erosion control or resistance. Geogrids and geotextiles are the common types of geosynthetics that are used to perform the reinforcement function. Woven and nonwoven geotextiles are the ones used commonly as filters, while geonets and nonwoven geotextiles are generally used to perform the drainage function. Geomembranes and GCL geocomposites are generally used to perform the containment (or liquid barrier) function. Geomats and certain woven geotextiles are used for erosion control/resistance applications.

The different functions and the mechanisms are discussed below.

6.4.1 Reinforcement

Reinforcement takes place when the geosynthetic (i.e., generally geotextile or geogrid) provides a tensile load–carrying element that modifies the stress-strain behavior of the system. In particular, the geosynthetic can complement soil that is good in compression but weak in tension. Typical examples of geosynthetic being used to perform the reinforcement function are in a flexible road pavement structure constructed on soft soil and construction of road embankments on soft soil. The mechanisms are illustrated in Figures 6.25 and 6.26.

The stress in the pavement structure due to the axle loading is typically distributed over a wider area by the base layer of the pavement. As shown in Figure 6.25, the vertical deformation and the membrane effect in the geosynthetic results in redistribution of the stresses in the underlying soil and tensile-stabilizing force developed in the geosynthetic. Consequently, the stress in the underlying soil is further reduced due to stress being distributed over an even wider area and the support provided by the vertical component of the geotextile tensile force. It is worth noting that in such applications, the geosynthetic must have adequate tensile strength, and its stress-strain characteristics must be compatible with the earth structure so that this strength is available at the design strains.

In the case of an embankment to be constructed on soft clay soil, a layer of geosynthetic (i.e., either a stiff geotextile or geogrid) is often used as basal reinforcement. Typically, a 0.3 to 0.5 m (1.0 to 1.6 ft) thick working platform of granular fill is initially

Figure 6.25 Road pavement on soft soil with geosynthetic reinforcement

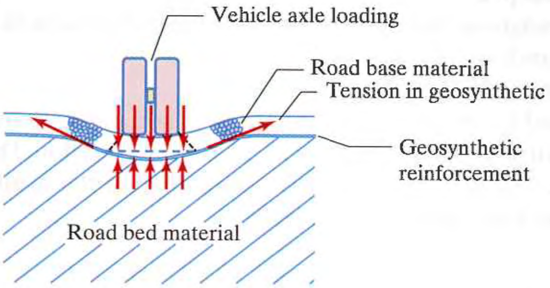
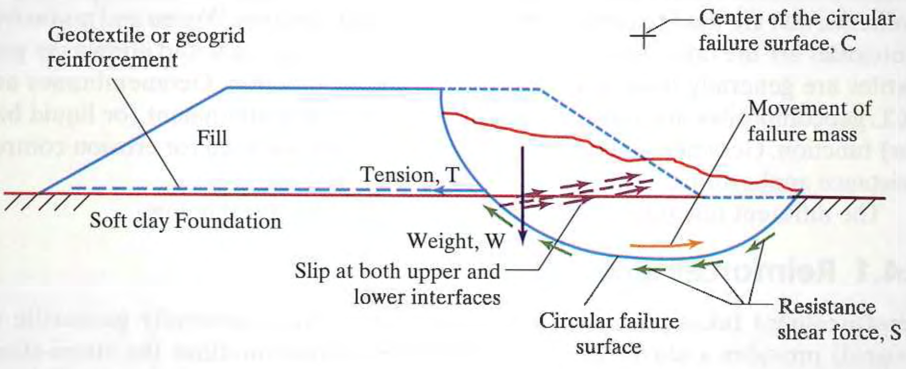


Figure 6.26 Slip circle analysis and reinforcement pullout failure of geosynthetic reinforced embankment on soft clay



constructed on the soft clay. The geosynthetic layer is then placed such that the higher strength and stiffness (usually the MD) of the geosynthetic is aligned toward the lateral direction (i.e., the weaker strength-stiffness or CMD of the geosynthetic is aligned in the longitudinal direction) of the embankment. Then the embankment fill material is added in small increments (e.g., 150–200 mm (6-8 in.) thickness) with each increment compacted using appropriate heavy machinery until the desired height is reached.

The stability of this type of embankment is commonly analyzed, using the limit equilibrium method considering circular type failure surfaces as shown in Figure 6.26. For such embankment, the short-term stability (i.e., stability immediately after constructing the embankment, commonly referred as undrained condition) is usually critical. Its stability condition will improve with time since the shear strength and compressibility characteristics of the soft clay foundation soil would improve with time when the excess pore water pressure that develops in it due to the embankment loading gradually dissipates. Such an embankment is designed with due consideration of the tensile force developed in the geosynthetic, which contributes to the stability of the embankment.

The weight of the embankment fill to the right of the circular potential failure surface (indicated as W in Figure 6.26) causes counterclockwise movement with respect to the center of the circle, C. This force tries to cause movement of the failure mass along the circular failure surface as shown, thereby resulting in the development of tensile force (T) in the geosynthetic reinforcement to resist that movement. There is shear resistance force (S) along the circular failure surface that also tries to keep the embankment stable. In a typical limit equilibrium analysis approach, the moment equilibrium of these three forces (W, S, and T) with respect to C is considered.

The tensile force (T) that is developed in the geosynthetic reinforcement is dependent on the geosynthetic–fill interface shear strength parameters c_{int} and ϕ_{int}, which are generally determined from interface direct shear tests (e.g., ASTM D5321) or pullout tests as discussed earlier in Section 6.3.2. The interface shear strength parameters are used to check whether adequate anchorage length (e.g., length L_a for the embankment on soft clay problem shown in Figure 6.27) is available to ensure that reinforcement would not get pulled out.

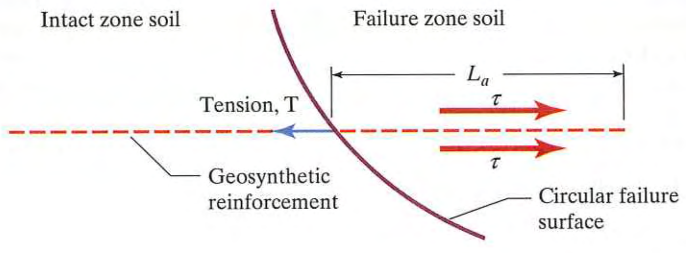
Checking for Reinforcement Pullout

The horizontal stresses and forces acting on the geosynthetic reinforcement are detailed in Figure 6.27. Considering the horizontal equilibrium of the forces acting on the geosynthetic reinforcement (for 1 m length of the embankment),

$$T = 2\tau L_a \tag{6.9}$$

Considerations for reinforcement anchorage length calculation

Figure 6.27



where

T = tensile force developed in geosynthetic reinforcement

τ = geosynthetic–fill interface shear resistance

L_a = anchorage length

As discussed in Section 6.3.2, at the verge of reinforcement pullout, the maximum value of τ could be estimated using $\tau_f = c_p + \sigma \tan \phi_p$. τ_f will vary along the anchorage length, depending on the normal stress (σ), but a reasonable estimate of τ_f could be made, using an average normal stress.

However, the shear resistance τ may not be uniformly distributed and fully mobilized (i.e., $\tau < \tau_f$) along the entire anchorage length L_a due to factors, such as stress concentration near the intersection of failure surface and geosynthetic reinforcement. One could introduce an efficiency factor E to account for such factors.

As is discussed in Section 6.5, a factor of safety (FS) is usually applied to the (laboratory measured) tensile strength (T) to account for installation damage, creep, and so forth, and T_{allow} is the value used in design (i.e., $T_{allow} = T/\text{FS}$).

Hence, the general design equation for estimating the anchorage length:

$$2\tau E L_a = T_{allow}\ (FS) \qquad \textbf{6.10}$$

where

E = efficiency factor

T_{allow} = allowable tensile force in geosynthetic

FS = factor of safety for the tensile strength of geosynthetic

It is noted that T_{allow} is used as the reinforcement tensile force for designing reinforced embankments, using the limit equilibrium method. The use of T (instead of T_{allow}) in the above equation for estimating L_a will yield a conservative estimate of the anchorage length. However, the FS in the above equation could be lower than that used for estimating the design reinforcement tensile force for limit equilibrium analysis.

EXAMPLE 6.2

A 4 m high geotextile-reinforced granular fill embankment on soft clay is to be designed, using slip circle limit equilibrium analysis method. The allowable tensile strength of the geotextile considered for use as reinforcement was estimated to be 60 kN/m, after allowing for the required FS. The geotextile–fill interface shear strength parameters were determined to be: adhesion = 0 and friction angle = 40°. Determine the anchorage length required behind the potential circular failure surface for mobilizing the allowable tensile strength of the candidate geotextile.

The approximate average normal stress on the geosynthetic could be assumed as $0.5 \times$ height of embankment $\times$ unit weight of fill $= 0.5 \times 4 \times 20 = 40$ kPa.

Hence, the $\tau_f = c_p + \sigma \tan \phi_p = 40 \tan 40^\circ = 33.6$ kPa

Using the design equation: $2\tau E L_a = T_{allow}(FS)$

Assuming an efficiency factor of 0.8 for the shear transfer and a FS of 2.0 for estimating the design tensile force for anchorage design,

$$2 \times 33.6 \times 0.8 \times L_a = 60 \times 2.0$$

Therefore, the anchorage length $L_a = 2.23$ m

Geogrid Reinforcement and Mechanism

As previously stated, geogrids perform only one of the major functions, that is reinforcement. For example, when granular fill material is placed over a geogrid in the construction of a road pavement base, the aggregate particles partially penetrate and project through the apertures and get locked between the ribs of the geogrid as illustrated in Figure 6.28. With loading from compaction machinery during construction, as well as vehicles on the road during service, the granular material tends to spread out. But, this is restrained by the geogrid, resulting in the development of tension, and the tensile strength of the geogrid becomes available to the granular material. This is commonly referred as interlocking mechanism.

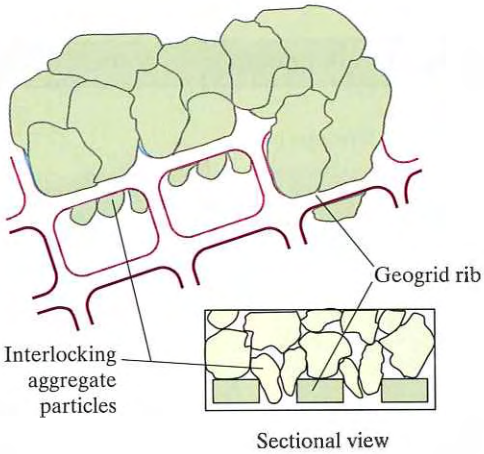
In addition to the provision of tensile supporting effect, geogrid reinforcement spreads the load and consequently improves the load distribution on the subgrade (i.e., natural in situ road bed material).

The properties of geogrids that relate to their use in reinforcement applications are wide-width tensile strength, single rib tensile strength, junction/joint strength, interface shear strength from the interface direct shear test, and anchorage strength from the pullout test.

The endurance properties of geogrids are important as they are used primarily in reinforcement applications. In particular, installation damage and tension creep behavior are important factors to be considered. Strength reductions in the order of 30 percent are possible due to installation damage, although higher strength loss values

Illustration of geogrid–granular fill interlocking mechanism

Figure 6.28



could occur where large, poorly graded, quarried aggregate is used and is placed and compacted by heavy construction equipment. Other factors, such as coarse soil impingement, falling objects, and other accidents may also cause a reduction in strength. For long-term applications, the tension creep or sustained load deformation is an important consideration. Reduction factors are commonly applied to determine the allowable strength to ensure failure of the system does not occur due to creep. As geogrids are used in reinforcement applications, the effects of high temperature can aggravate the strains arising from creep. For high-temperature applications, testing should be carried out to determine the suitability of particular geogrids for a specific application.

Typical Uses of Geogrids

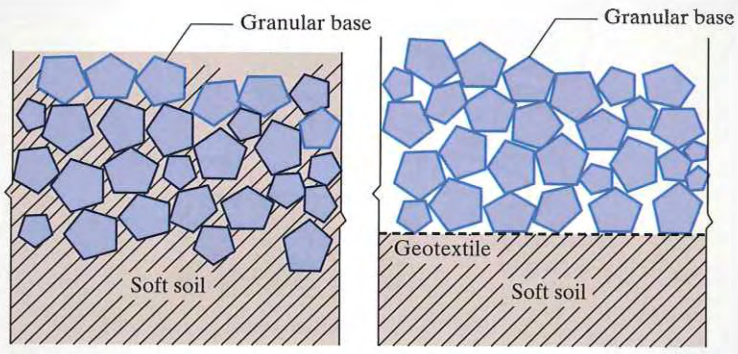
Typical uses of geogrids include beneath aggregate in unpaved roads and railways and beneath surcharge fills or temporary construction sites. They are used to reinforce embankment fills and earth dams, as gabions for wall construction, and as basal reinforcement over soft soils or between pile foundation caps. Geogrids bridge over cracked or jointed rock, form mattresses for fills over soft soils, and are used as asphalt reinforcement in pavements.

6.4.2 Separation

Separation is achieved by placing a geosynthetic between two different materials (e.g., gravel and soft clay) so that the integrity and functioning of each material will remain the same. Geotextiles are the only type of geosynthetics commonly used for separation. A typical case where a geotextile could be used as a separator is between the granular base of a road constructed on soft ground (e.g., soft clay foundation soil) where it prevents intermixing of the two materials and migration of one material into the other. This is illustrated schematically in Figure 6.29. Strength, pore size, and permeability are the important properties of geotextiles considered when they are used for separation.

When a geotextile is used for the separation function as shown in the figure, the deformations in the geotextile fabric are localized and occur around individual stone particles. While the strength of the geotextile is not directly related to the separation function, the chosen fabric must survive the possible installation damage and the design load conditions in order to be effective in the separation task. The required

Figure 6.29 Illustration of a geotextile functioning as a separator



geotextile strength properties are good CBR plunger burst strength, tensile strength, puncture and impact resistance (e.g., drop cone puncture test), and tear resistance (e.g., trapezoidal tear resistance test).

6.4.3 Filtration

As discussed in Section 6.3.3, in particular with reference to Figure 6.18, filtration is the process in which the geotextile is used to allow water to pass through while preventing the passage of soil (solid) particles. Therefore, to perform as a filter, the geotextile requires adequate cross-plane permeability to allow the movement of water, which requires an open fabric structure. At the same time, the geotextile must retain soil, requiring a tight fabric structure.

A geotextile must also have a long-term flow compatibility with the soil so that it will not excessively clog (i.e., soil particles blocking the pores or getting trapped within the geotextile fibers) during its intended life.

Mechanisms of Filtration

As the requirement of a geotextile filter is to allow fluid to flow and prevent soil movement, the mechanism of filtration involves the formation of fabric-induced soil filter. Mechanisms for long-term soil-to-geotextile fabric flow compatibility proposed by McGown (1978) are illustrated in Figure 6.30. In the mechanism illustrated in Figure 6.30a, a bridging network is formed upstream adjacent to the geotextile, and a soil filter gets formed further upstream. In the mechanism detailed in Figure 6.30b, soil solid particles arch on the upstream side of geotextile openings, thereby aiding natural filtration.

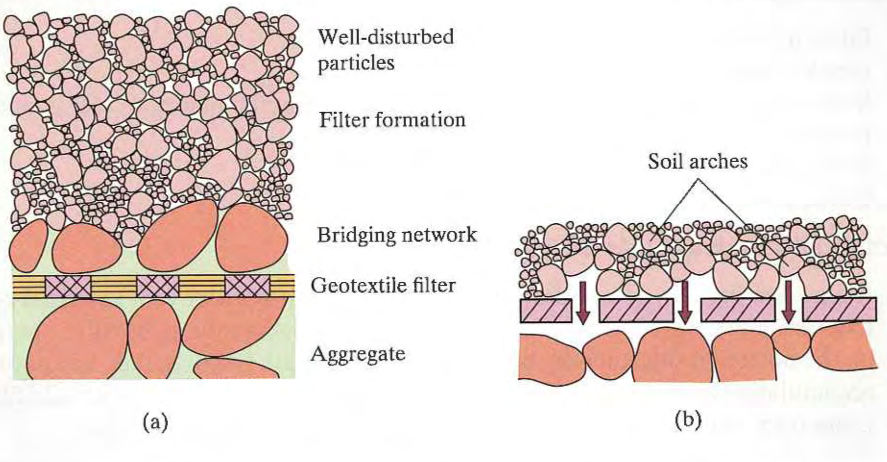
Types of Flow

Steady unidirectional flow in drainage filters (e.g., geotextile filter around a perforated drainage pipe in a trench drain beside a road pavement).

Reversing flow—moderate cycle (e.g., river and coastal defense filters)

Hypothetical mechanisms involved in long-term soil-to-geotextile fabric flow compatibility: (a) formation of an upstream soil filter (b) upstream soil articles arching over geotextile opening after McGown, 1978

Figure 6.30



(Based on McGown, A. (1978). The properties of nonwoven fabrics presently identified as being important in public works applications. Index 78 Programme, University of Strathclyde, Glasgow, Scotland.)

Reversing flow—very short cycle (e.g., anti-pumping filters beneath railway ballast)

Requirements for Geotextile Filters

The requirements are similar to granular filters

- The opening must be large enough to allow water to pass through without pore water pressure (pwp) buildup in the upstream soil. This is commonly referred to as the permeability criterion.
- The opening must be small enough to prevent or minimize the passage of soil particles (to maintain structural integrity, referred as the soil retention criterion.

High cross-plane permeability and adequate pore size distribution are the properties required for effective filtration.

Long-Term Flow Compatibility Over time, a number of hypothetical mechanisms may form in the system that will determine the soil-to-fabric flow characteristics (e.g., as illustrated in Figure 6.30). In some situations, the geotextile might get excessively clogged with either soil particles or suspended solids from the ground water passing through it. This usually occurs with poorly graded fine, cohesionless soils, such as loess, rock flour, and stone quarry fines; dispersive clays that separate into fine particles over time; high suspended solids in the permeating water, such as turbid river water (Koerner, 2005).

Soil Retention Criteria Simple design guidelines proposed by AASHTO (1991):

- For upstream soil with $\leq 50\%$ passing the 0.075 mm opening size (US Standard No. 200 sieve), the O_{95} of the geotextile filter fabric should be less than 0.60 mm (i.e., AOS of the fabric $\geq$No. 30 US Standard sieve)
- For upstream soil with $\geq 50\%$ passing the No. 200 sieve, O_{95} of the geotextile filter fabric should be less than 0.30 mm (i.e., AOS of the fabric $\geq$No. 50 US Standard sieve)

Carroll (1983) recommendation:

$$O_{95} < (2 \text{ or } 3)\, d_{85}$$

where d_{85} is the upstream soil particle size in mm for which 85% of the soil is finer.

Advantages of Geosynthetic Filters

- Filtering ability is factory controlled.
- Quick visual inspection for quality assurance.
- More easily transported to construction site than granular materials (i.e., economic benefit).
- Filter has an independent tensile strength.
- Quick and easy installation and less labor cost.

Potential Disadvantages of Geosynthetic Filters

- Long-term life of a geosynthetic in soil environment is unproven. Potential problems are clogging, blocking (i.e., solid particles blocking the geotextile openings on the upstream side, thereby blocking water passage), blinding (i.e., fine particles accumulating and forming a layer upstream, thereby blinding the geotextile filter).
- Long-term exposure to UV light could seriously affect the geotextile.

- Tearing and other installation damage to the geotextile, improper overlap, and so forth. Therefore, care is needed during construction.

Typical Geotextile Filter Applications Applications for geotextile filters include placement beneath stone bases for road and airfield pavements, around stone and/or perforated pipe in subsoil drains, as a silt fence or curtain, between backfill soil and drains in retaining walls, and with geonets to prevent soil intrusion.

6.4.4 Drainage

As discussed in Section 6.3.3 with reference to Figure 6.20, drainage is the process in which the geosynthetic is used to transmit fluid (generally water) in the plane of its structure. Geotextiles, geonets, and their composites are the types of geosynthetics used for the drainage function. Their in-plane permeability, which depends on thickness and voids available to transmit water, is an important consideration for drainage function. The soil particles that get transmitted with the water may reduce the voids available for water transmission and consequently affect the long term performance. The discussion on filtration concerning long-term flow compatibility of the soil with the geotextile and soil retention considerations apply for the drainage function as well.

Geotextiles in Drainage Applications

A geotextile should have high in-plane permeability to perform the drainage function. However, its in-plane permeability is dependent on the thickness, which in turn depends on the normal stress acting on it. Hence, good drainage is typically provided by thick or multilayered (needle-punched, nonwoven) geotextiles with high in-plane permeability that resists compression and also have good long-term flow compatibility with the soil to avoid excessive clogging.

Typical uses of geotextiles in drainage include as drains beneath railway ballast, as drains behind retaining walls, as drains for roof gardens, as pore water dissipators in earth fills, and as replacements for sand drains.

Geonets

As the primary function of geonets is in-plane drainage, the in-plane hydraulic flow rate, or transmissivity, is the major property of geonets. Other properties do, however, influence the flow rate of geonets over their service life; these include:

Tensile and Shear Strength—If the geonet is used where soil deformation or movement could occur, it must have adequate tensile and shear strength to avoid damage.

Compressive Strength—Geonets must have adequate cross-plane compressive strength as compressive deformation will adversely affect the geonet's ability to conduct liquid within its planar structure.

Endurance of Geonets

The following factors need to be considered:

- *Type of polyethylene resin used.* High-density resins give high strength, compression resistance, and creep resistance. Low-density resins give more flexibility and better stress-crack resistance.
- *Creep behavior.* Geonets must be able to resist excessive creep under sustained, long-term, compressive loads. Excessive creep will result in a reduction in thickness and will adversely affect the flow rate of liquid in the geonet plane.

- *Intrusion of adjacent materials*. Geonets are generally covered on both sides to prevent the intrusion of soil or other material into their structure. Intrusion of the covering material into the geonet membrane must also be considered. If the covering material, usually a geotextile, is forced under an applied load to occupy some of the open spaces within the geonet structure, there is a reduction of in-plane flow, as the in-plane transmissivity of geotextiles are lower than that of geonets.

Durability of Geonets

The durability properties of geonets are similar to other types of geosynthetics as they are made from the same materials.

Testing of geonets: Although geonets are not designed for reinforcement applications, the strength properties are important to ensure that they survive in order to carry out their intended functions. The hydraulic properties are important to test as the in-plane drainage properties of geonets are vital to their application.

Key tests include: wide-width tensile strength, compressive strength, shear strength, and in-plane flow rate (transmissivity).

Typical uses of geonets include: drainage behind retaining walls, drainage on seeping rock slopes, drainage in frost-susceptible soils, drainage beneath building foundations, leachate collection in landfills, underdrain systems beneath landfills, drainage blankets beneath surcharge fill.

6.4.5 Containment

As discussed in Section 6.2.5, geomembranes and GCLs are geosynthetics with very low permeability that are used for containment (i.e., as the barrier for the migration) of fluid and gas (vapor). There are wide-ranging geomembrane–geotextile composites and GCLs that are produced for containment applications to facilitate easy construction, different site conditions, and improved performance. Obviously, all of them cannot be discussed in a book chapter. Only the important details of geomembranes are dealt with here, and the reader should refer to other sources of information for further learning.

Functions and Properties of Geomembranes

The major function of geomembranes is a barrier to liquid or vapor flow or migration. The major physical properties that influence the barrier function are:

Thickness and density—The physical dimensions of the geomembrane are important to its ability to act as a liquid or vapor barrier. While no polymeric material is 100 percent impervious, the thicker and denser the material, the better barrier to liquid and vapor it is. Quality control concerning this is important to ensure local weaknesses in the membrane do not occur.

Water-vapor transmission and permeability—The liquid-vapor transmission and permeability properties of the geomembrane are dependent, among other things, on the polymeric material (for example, HDPE) that the membrane is manufactured from. Typically, the permeability of a geomembrane ranges between 1×10^{-12} and 1×10^{-15} m/S, which is about three orders of magnitude lower than clay. However, the challenge is to ensure that the geomembrane does not get many holes in it during service (such as damage of geomembrane during construction or leakage at the weld seam), which could result in significant leakage.

Tensile test on sheet and shear and peel tests on seams—Geomembrane is often supplied in rolls of certain width, and they are welded at the site to the required size and shape. As the geomembrane must be able to survive both construction and loading within the system it is designed for, there is a requirement for it to possess the minimum strength properties to survive and to perform its intended function as a barrier. The tensile properties of geomembranes are typically determined in accordance with ASTM D6693 and ASTM D4885. The geomembrane seams are also tested for shear strength and peel, adopting methods such as ASTM D6392.

Tear, impact, and puncture resistance—As previously stated, the geomembrane must survive construction as well as local stresses such as falling rocks, being laid over rough ground, and so forth, in order to perform its barrier function.

Endurance and durability of geomembranes—The endurance properties of geomembranes are similar to those of other types of geosynthetics as they are made from the same polymer materials. Of particular importance for geomembranes is their endurance to UV, chemical, and biological attack. As geomembranes are often used as barriers for contaminated waste or water, they must have a good resistance to attacks from these substances in order to survive the entire design life. The effects of UV, chemical, and biological degradation must be considered, as more than one of these is usually present in any application.

It is noted that in most applications, a protective geotextile layer is placed on either side of a geomembrane in order to reduce the likelihood of damage.

Testing of Geomembranes

Testing of geomembranes includes basic strength properties to ensure their survivability, as well as application-specific tests. These include testing a range of reagents likely to be encountered to determine their effect on the geomembrane. Testing is often carried out at elevated temperatures to accelerate the procedures.

Typical Uses of Geomembranes

Typical uses of geomembranes include liners for potable water, liners for liquid waste, liners for canals, liners for solid waste landfills, liners for vertical walls, and barriers to liquid and vapors beneath buildings.

6.4.6 Erosion Control

As discussed in Section 6.2.7, geomats are commonly used for controlling soil erosion from rain drops and runoff. Geomats are net-like open-structured geosynthetics that allow natural vegetation with time. Hence, biodegradable geomat products are also available in the market.

Typical applications are soil erosion protection of steep slopes; river bank protection, particularly around the water level that may fluctuate due to wave action; and general landscaping.

6.4.7 Typical Examples of Geosythetics Use in the Field

Geosynthetics are used widely in civil engineering projects, such as in the construction of road embankments, steep slopes, unpaved roads on soft ground, liners of landfills, and filtration and drainage behind retaining structures. Some typical construction activities involving geosynthetics products are shown in Figures 6.31 to 6.34 below.

Figure 6.31 A layer of geogrid reinforcement is being laid for constructing a road embankment over swamp area.

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 6.32 Geosynthetic prefabricated vertical drains (PVDs) are being installed to stabilize soft soil for constructing a road embankment.

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

6.5 Selection of Geosynthetics

As seen previously, there are different types of geosynthetics, and within each type, there is a variety of available products. Typical ranges of the properties of different types of geosynthetics are summarized in Table 6.5. Selecting the most appropriate geosynthetic for a particular application is an important objective of the design of the

Geotextiles are used for separation and filtration/drainage functions in the construction of a road embankment.

Figure 6.33

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

A geotextile filter is being laid for constructing a subsurface drain for a road.

Figure 6.34

(Courtesy of N. Sivakugan, James Cook University, Australia)

structure. The selection is commonly based on various considerations, including how the main function the geosynthetic is expected to perform in the civil engineering structure, properties of the geosynthetic that are needed for performing that function, properties of the available geosynthetics that match those needs, and the relative cost (such as a cost-benefit ratio type of analysis). Important aspects of the selection of a geosynthetic for a particular application are discussed in this section.

The main function and the properties that are important or required for performing that function are related. Hence, knowing the properties that are relevant for each

Table 6.5 Typical range of properties of common geosynthetics

Types of geosynthetics	Tensile strength (kN/m)	Extension at max. load (%)	Apparent opening size (mm)	Water flow rate (volume permeability) (liters/m²/s)	Mass per unit area (g/m²)
Geotextiles					
Nonwovens					
Heat-bonded	3–25	20–60	0.02–0.35	10–200	60–350
Needle-punched	7–90	30–80	0.03–0.20	30–300	100–3000
Resin-bonded	5–30	25–50	0.01–0.25	20–100	130–800
Wovens					
Monofilament	20–80	20–35	80–2000	80–2000	150–300
Multifilament	40–1200	10–30	20–80	20–80	250–1500
Flat tape	8–90	15–25	5–25	5–25	90–250
Knitteds					
Weft	2–5	300–600	0.20–2.0	60–2000	150–300
Warp	20–800	12–30	0.40–1.5	80–300	250–1000
Stitch bondeds	30–1000	10–30	0.07–0.5	50–100	250–1000
Geogrids					
Extruded	10–200	20–30	15–150	NA	200–1100
Textile-based					
Knitted	20–400	3–20	20–50	NA	150–1300
Woven	20–250	3–20	20–50	NA	150–1100
Bonded cross-laid strips	30–200	3–15	50–150	NA	400–800
Geomembranes					
Natural					
Reinforced (made from bitumen and nonwoven geotextile)	20–60	30–60	0	0	1000–3000
Plastomeric (made from plastomers such as HDPE, LDPE, PP or PVC)					
Unreinforced	10–50	50–200	0	0	400–3500
Reinforced	30–60	15–30	0	0	600–1200

Elastomeric (made from elastomers, i.e., rubbers of various types)					
Reinforced	30–60	15–30	0	0	500–1500
Geocomposites					
Geosynthetic clay liners	10–20	10–30	0	0	5000–8000
Linked structures (geostrip based)[1]	100–1500	3–15	NA	NA	400–4500

Notes: NA is not applicable

[1] *Geostrips are geocomposites having tensile strength in the range 20–200 kN and extension at maximum load in the range 3–15%*

(Based on information compiled by Lawson C.R. and Kempton, G.T. (1995). Geosynthetics and their use in reinforced soil. Terram Ltd., Pontypool, Gwent, U.K. and Shukla, S.K. and Yin, J-H. (2006). Fundamentals of geosynthetic engineering. Taylor and Francis Publishers, U.K.)

function is helpful for selecting the geosynthetic for an application. The relationship between the function, type of geosynthetic commonly used for performing such function, and important properties that need consideration is presented in Table 6.6.

6.5.1 Available versus Required Property

Geosynthetics, being made of polymeric materials that are used in varying soil and environmental conditions, have properties that are affected by factors such as UV and thermal degradation, damage during installation and construction, creep and stress relaxation, and chemical and biological degradation during its service life. Consequently, the properties of a geosynthetic, that are usually determined from laboratory testing on samples of freshly manufactured product, would decrease or change

Relationship between the function, type of geosynthetic commonly used, and important properties

Table 6.6

Function	Geosynthetic type commonly used	Important properties of geosynthetic
Reinforcement	Geotextile, geogrid	Tensile strength, stiffness (or modulus), geosynthetic–soil interaction properties (interface shear, interlocking and pullout behavior characteristics), creep, relaxation, durability
Separation	Geotextile	Tensile strength, burst strength, tear strength, AOS, grab strength, puncture resistance, durability
Filtration	Geotextile	Permittivity, AOS, geotextile–upstream soil compatibility for non-clogging, durability
Drainage	Geotextile, geonet	Transmissivity, AOS, geosynthetic–soil compatibility for non-clogging, durability
Containment (fluid barrier)	Geomembrane, GCL	Cross-plane permeability, tensile strength, geosynthetic–fluid compatibility, durability

unfavorably with time during the service life of the structure due to these factors. The property of a geosynthetic at a particular time during its use in a civil engineering structure is referred to as the available property for performing the intended function, and its variation with time is illustrated in Figure 6.35.

On the other hand, the geosynthetic is expected to perform a particular primary function when it is used in a civil engineering structure, and this is related to a certain property (or properties) of the geosynthetic, hereafter referred to as the functional property. Therefore, for satisfactory performance of the structure from its construction until the end of its service life, the geosynthetic is required to have a certain value for that functional property. The requirement of the structure for that functional property varies with time, and this variation is also illustrated in Figure 6.35.

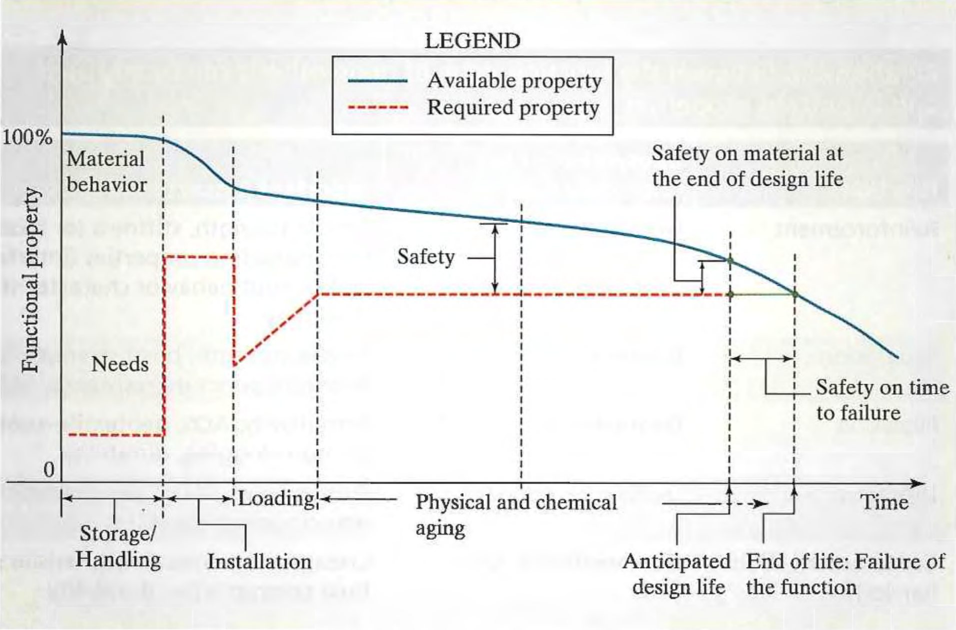
The difference between the available property and the required property indicates the safety margin, which varies with time. The geosynthetic for a particular application is chosen such that the structure will have sufficient margin of safety throughout its anticipated design life.

The numerical value of the available property over the design life is commonly referred to as the allowable property, and it is estimated by applying different reduction factors for the values determined from laboratory tests on samples of geosynthetics or from the values reported by the manufacturers in their product literature. Typically,

$$\text{Allowable functional property} = \frac{\text{Test functional property}}{RF_{D1} \times RF_{D2} \times RF_{D3} \times \ldots} \qquad \textbf{(6.11)}$$

where RF_{D1}, RF_{D2}, RF_{D3}, ... are reduction factors for different degradation considerations $D1$, $D2$, $D3$, and so forth. These reduction factors should correspond to the

Figure 6.35 Typical available and required (design) value of a property as a function of time



(Based on Standard Australia (2002). Technical Handbook: Geosynthetics – Guidelines on durability, Standard Australia Int. Ltd, Sydney, NSW 2001. From HB 154-2002.)

function (such as reinforcement, separation, etc.), geosynthetic type (e.g., geotextile or geogrid), and its application in the field, such as unpaved roads, embankments, and so forth.

For example, the allowable tensile strength for a reinforcement application of a geosynthetic is determined using the equation:

$$\text{Allowable tensile strength, } T_{allow} = \frac{\text{Ultimate tensile strength, } T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CBD}} \quad \textbf{(6.12)}$$

where RF_{ID} = reduction factor for installation damage; RF_{CR} = reduction factor for creep, and RF_{CBD} = reduction factor for chemical and biological degradation. These factors are dependent on the civil engineering structure (for example, unpaved road, railroad, slope stabilization, etc.) and its design life. They are generally estimated in accordance to the relevant design standard(s) and guideline(s)—for example, AASHTO, Department of Transport design standard/guideline, and ASTM. Typical ranges of strength reduction factors for reinforcement and separation functions are given in Table 6.7.

For flow-related applications (i.e., for filtration and drainage functional use) of geotextiles, the allowable flow rate is estimated from the ultimate flow rate obtained from laboratory test or manufacturer's product literature using the equation:

$$\text{Allowable flow rate, } q_{\text{allow}} = \frac{\text{Ultimate flow rate, } q_{\text{ult}}}{RF_{IN} \times RF_{BC} \times RF_{CC} \times RF_{CR} \times RF_{SCB}} \quad \textbf{(6.13)}$$

where RF_{IN} = reduction factor for adjacent material intruding into the void space,
RF_{BC} = reduction factor for biological clogging,
RF_{CC} = reduction factor for chemical clogging,
RF_{IN} = reduction factor for creep reduction of void space, and
RF_{SCB} = reduction factor for soil clogging and blinding of the geotextile.

Recommended flow rate reduction factors for filtration and drainage functions of geotextiles are given in Table 6.8.

Recommended values of strength reduction factors for determining the allowable tensile strength

Table 6.7

Application area	Values of reduction factors					
	Geotextile			Geogrid		
	RF_{ID}	RF_{CR}	RF_{CBD}	RF_{ID}	RF_{CR}	RF_{CBD}
Separation	1.1–2.5	1.5–2.5	1.0–1.5	N/A	N/A	N/A
Unpaved roads	1.1–2.0	1.5–2.5	1.0–1.5	1.1–1.6	1.5–2.5	1.0–1.6
Paved roads	1.1–1.5	1.0–2.0	1.0–1.5	1.2–1.5	1.5–2.5	1.1–1.7
Embankments	1.1–2.0	2.0–3.5	1.0–1.5	1.1–1.4	2.0–3.0	1.1–1.5
Slope stabilization	1.1–2.0	2.0–3.0	1.0–1.5	1.1–1.4	2.0–3.0	1.1–1.5
Reinforced soil walls	1.1–2.0	2.0–4.0	1.0–1.5	1.1–1.4	2.0–3.0	1.1–1.5
Foundations	1.1–2.0	2.0–4.0	1.0–1.5	1.2–1.5	2.0–3.0	1.1–1.6

(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

Table 6.8 **Recommended values of flow reduction factors for geotextiles**

Application area	Values of reduction factors				
	RF_{IN}	RF_{BC}	RF_{CC}	RF_{CR}	RF_{SCB}
Retaining wall filters	1.0–1.2	1.0–1.3	1.0–1.2	1.5–2.0	1.0–1.3
Underdrain filters	1.0–1.2	2.0–4.0	1.2–1.5	1.0–1.5	2.0–4.0
Erosion control filters	1.0–1.2	2.0–4.0	1.0–1.2	1.0–1.5	2.0–4.0
Landfill filters	1.0–1.2	2.0–5.0	1.2–1.5	1.5–2.0	2.0–5.0
Gravity drainage	1.0–1.2	1.2–1.5	1.2–1.5	2.0–3.0	1.2–1.5
Pressure drainage	1.0–1.2	1.1–1.3	1.1–1.3	2.0–3.0	1.1–1.3

(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

Similarly for drainage applications of geonets,

Allowable flow rate, $q_{\text{allow}} = \dfrac{\text{Ultimate flow rate, } q_{ult}}{RF_{IN} \times RF_{BC} \times RF_{CC} \times RF_{CR}}$; and the terms are as defined above for drainage applications of geotextiles. Recommended flow rate reduction factors for drainage function of geonets are given in Table 6.9.

Table 6.9 **Recommended values of flow reduction factors for geonets**

	Values of reduction factors			
Application area	RF_{IN}	RF_{BC}	RF_{CC}	RF_{CR}
Retaining walls and soil slopes	1.3–1.5	1.0–1.5	1.1–1.5	1.2–1.4
Drainage blankets	1.3–1.5	1.0–1.2	1.0–1.2	1.2–1.4
Landfill covers	1.3–1.5	1.5–2.0	1.0–1.2	1.1–1.4
Landfill leachate collection systems	1.5–2.0	1.5–2.0	1.5–2.0	1.4–2.0
Roof and plaza decks	1.2–1.4	1.1–1.3	1.0–1.2	1.0–1.2

(Based on Koerner, R. M. (2005). Designing with geosynthetics – Fifth Edition, Pearson Prentice Hall publishers, USA.)

EXAMPLE 6.3

A PET geotextile is being considered for use as reinforcement in a road embankment over soft clay, and its ultimate tensile strength determined from wide strip tensile testing is 216 kN/m. Determine the allowable tensile strength of the geotextile for the design of this reinforced embankment.

Solution:

The value of each reduction factor is selected based on the service life of the structure and relevant design standard. From Table 6.7, the reduction factors for installation damage, creep, and chemical and biological degradation range between 1.1–2.0, 2.0–3.5 and 1.0–1.5 respectively.

Assuming that the reduction factor for installation damage = 2.0; reduction factor for creep = 3.0, and reduction factor for chemical and biological degradation = 1.2

$$T_{allow} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CBD}} = \frac{216}{2.0 \times 3.0 \times 1.2} = 30.0 \text{ kN/m}$$

6.5.2 Selection of a Geosynthetic

Geosynthetics are used in a particular civil engineering application to perform certain main function for a minimum expected time period, called the design life. Typical design life is:

- Less than 5 years for short-term use
- Around 25 years for temporary use
- Fifty to 100 years or more for permanent use (e.g., 70 years for a reinforced soil wall and 100 years for a reinforced soil bridge abutment foundation)

The design of a geosynthetic incorporated structure is aimed to ensure its stability, integrity, and serviceability over its intended design life.

The primary function of the geosynthetic is determined, and a numerical value is calculated for the particular property that is required for performing that function. This is referred to as the required or design value of functional property. Relevant factor of safety can then be applied and a suitable geosynthetic chosen. The steps involved in geosynthetic selection are:

1. Assess the particular application and identify the primary function of the geosynthetic (e.g., reinforcement, filtration, drainage, separation, etc.).
2. Estimate the design life of the geosynthetic incorporated structure or geosynthetic application.
3. Determine the required functional property of the geosynthetic for the primary function (e.g., strength, permittivity, transmissivity, etc.) for the particular application (for the entire design life). This is usually done through geotechnical analysis, which is beyond the scope of this book. The reader should refer to books or other sources specializing on design using geosynthetics.
4. Select the geosynthetic type and a candidate geosynthetic that could satisfy the requirement.
5. Obtain the available property at the end of design life (i.e., the allowable functional property) of the chosen candidate geosynthetic (e.g., obtain the property value by testing or from manufacturer's literature and apply the relevant reduction factors recommended in design standards).
6. Calculate the factor of safety (FS):

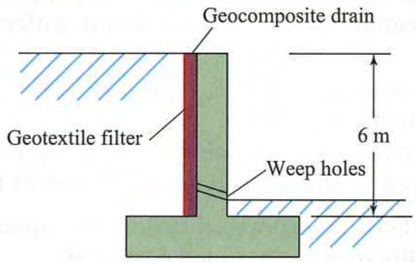
$$\text{FS} = \frac{\text{Allowable functional property}}{\text{Required (or design) functional property}}$$

7. Check whether the FS is acceptable.
8. If the chosen geosynthetic is acceptable, check whether there is any other function of the geosynthetic that is also critical. If yes, repeat the above steps.
9. Try different geosynthetics, assess the cost-benefit ratio, and select the most appropriate geosynthetic.

EXAMPLE 6.4

A geotextile filter is being considered to protect the geocomposite drain behind a retaining wall as shown in Figure 6.36. The wall stem is 6 m high, retaining a silty clay soil with permeability, $k = 7.5 \times 10^{-5}$ *m/s*; particle size corresponding to 50 percent passing, $d_{50} = 0.05$ mm; particle size corresponding to 85 percent passing, $d_{85} = 0.18$ mm; coefficient of uniformity, $C_u = 4.8$; and relative density, $D_R = 85\%$. The rate of flow into the drain under the critical design condition of water level being at the surface of the retained soil has been estimated from geotechnical analysis as 33.5×10^{-5} m^3/s/m. The candidate geotextile is a heat-bonded nonwoven with a permittivity, $\psi = 0.01$ s^{-1} with an $AOS = 0.21$ mm. Check the adequacy of the geotextile for flow and soil retention.

Figure 6.36 Schematic of the cantilever wall and its drainage system



Solution:

Given that the design or required flow rate, $q = 33.5 \times 10^{-5}$ m³/s/m

According to Darcy's law, $q = k\ i\ A = k\dfrac{\Delta h}{t}A$, where Δh = head loss and t = thickness of the geotextile.

Since the critical design water level is at the surface, $\Delta h = 6.0$ m

By definition: permittivity, $\psi = k/t$

Therefore, the required permittivity,

$$\psi_{req} = \frac{q}{\Delta h\ A} = \frac{33.5 \times 10^{-5}}{7.5 \times (7.5 \times 1.0)} = 0.596 \times 10^{-5}\ \text{s}^{-1}$$

Check against the permittivity of the geotextile:

$$\psi_{ult} = 0.01\ s^{-1}$$

$$\text{Allowable permittivity, } \psi_{allow} = \frac{\text{Ultimate permittivity, } \psi_{ult}}{RF_{IN} \times RF_{BC} \times RF_{CC} \times RF_{CR} \times RF_{SCB}}$$

Selecting the reduction factors from Table 6.8 as $RF_{IN} = 1.2$, $RF_{BC} = 1.3$, $RF_{CC} = 1.2$, $RF_{CR} = 2.0$ and $RF_{SCB} = 1.3$, $\psi_{allow} = \dfrac{0.1}{1.2 \times 1.3 \times 1.2 \times 2.0 \times 1.3} = 0.00205\ \text{s}^{-1}$

Factor of safety, FS $= \dfrac{\psi_{allow}}{\psi_{req}} = 0.00205/(0.596 \times 10^{-5}) = 345$

Therefore, this particular geotextile is acceptable for flow.

Check for soil retention:

AOS of geotextile = 0.21 mm
According to Carroll's soil retention criterion, $O_{95} < 2.5 \times d_{85}$
For the upstream retained soil, $2.5 \times d_{85} = 2.5 \times 0.18$ mm = 0.45 mm
For the candidate geotextile, $AOS = O_{95} = 0.21$ mm <0.45 mm
The geotextile is acceptable and FS = 0.45 / 0.21 = 2.14

EXAMPLE 6.5

For the retaining wall detailed in Example 6.4, a geocomposite with an ultimate transmissivity of 0.2 $m^3/min/m$ (or m^2/min) is being considered for the drainage function. Determine the FS with respect to flow for this geocomposite.

Solution:

The flow gradient within the geocomposite, $i = \sin\ 90° = 1.0$

Calculate the required transmissivity, θ_{req}

According to Darcy's law, $q = k\ i\ A = k\ i\ (t \times l)$

[Note: considering 1 m length of wall]

$$\text{Therefore, } \theta_{req} = k\ t = \frac{q}{i\ (1)} = \frac{33.5 \times 10^{-5}}{1.0 \times 1} = 33.5 \times 10^{-5}\ \text{m}^2/\text{s}$$

$$\text{Allowable transmissivity, } \theta_{allow} = \frac{\text{Ultimate transmissivity, } \theta_{ult}}{RF_{IN} \times RF_{BC} \times RF_{CC} \times RF_{CR}}$$

Selecting the reduction factors from Table 6.9 as $RF_{IN} = 1.5$, $RF_{BC} = 1.5$, $RF_{CC} = 1.5$, $RF_{CR} = 1.4$, $\theta_{allow} = \dfrac{0.2/60}{1.5 \times 1.5 \times 1.5 \times 1.4} = 7.054 \times 10^{-4}\ \text{m}^2/\text{s}$

Factor of safety, $FS = \dfrac{\theta_{allow}}{\theta_{req}} = 7.054 \times 10^{-4}/33.5 \times 10^{-5} = 2.1$ which is acceptable.

6.6 Summary

1. Geotextiles, geogrids, geonets, geomembranes, geomats, and various geocomposites are the different types of geosynthetics commonly used in civil engineering applications. They are manufactured of polymers, namely polyethylene (PE), polypropylene (PP), polyvinyl chloride (PVC), polyamide (PA), polyester (PET), and polystyrene (PS).
2. Geosynthetics are used to perform one or more major functions: reinforcement of soils to support tension, separation of dissimilar soils (e.g., separate good granular fill from soft clay), filtration of soil particles from flowing water, drainage of water and other fluids, containment (or liquid or gas barrier), and erosion control or protection.
3. Tensile strength, modulus, puncture resistance, tear strength, burst strength, creep and stress relaxation behavior, cross-plane permeability, in-plane permeability and AOS of geosynthetic; soil-geosynthetic interaction properties (e.g., interface shear, interlocking, geosynthetic pullout behavior); and geosynthetic-soil compatibility for clogging are some of the important properties that are usually determined from laboratory tests.

4. Properties of geosynthetics change with time mainly due to UV and thermal degradation, damage during installation and construction, creep and stress relaxation, and chemical and biological degradation during its service life.
5. Geotextiles is the largest group among geosynthetics, manufactured of different types of polymeric fibers, using mostly woven or nonwoven methods. They are used mainly for reinforcement, filtration, drainage, and separation.
6. Geogrids and geotextiles are the common types of geosynthetics used as reinforcement. The tensile properties (strength and modulus) and the geosynthetic-interface interaction properties (i.e., interface shear, interlocking and pullout behavior characteristics) are most important considerations for the reinforcement function. Installation damage, creep, and chemical and biological degradation are usually accounted for when determining the allowable tensile strength for design.
7. Geotextiles are the only type of geosynthetic used for filtration function application and cross-plane permeability (or permittivity), AOS, geotextile–upstream soil compatibility for non-clogging, and durability are the important considerations. Intrusion of adjacent material into the void space, creep reduction of void space, chemical clogging, and biological clogging are usually accounted for when determining the allowable filtration properties for design.
8. Geotextiles and geonets are the types of geosynthetics commonly used for drainage function application, and their in-plane permeability (or transmissivity), AOS, and geotextile–upstream soil compatibility for non-clogging and durability are the important considerations. Intrusion of adjacent material into the void space, creep reduction of void space, chemical clogging, and biological clogging are usually accounted for when determining the allowable drainage properties for design.
9. Geomembranes and GCLs are the types of geosynthetics used for liquid barrier (or containment) applications, and their cross-plane permeability, tensile strength, geosynthetic–fluid compatibility, and durability are the important considerations.
10. For selecting the geosynthetic for an application, the primary function of the geosynthetic is identified, and it is followed by geotechnical analysis to determine the required functional property of the geosynthetic for that function (for example, strength, permittivity, transmissivity, etc.). Selection of the geosynthetic is based on the estimation of the allowable property by applying appropriate reduction factors for the test value and comparing it with the required property.

Exercises

1. Geosynthetic made of which type of polymer is suitable for use in construction below water so that it will not float?
   a. Polyethylene
   b. Polypropylene
   c. Polyester
   d. All of the above

2. A woven geotextile made of which type of polymer fiber will have the lowest extension at break?
   a. High-density polyethylene
   b. Polyester
   c. Polypropylene
   d. Nylon

3. Which type of geosynthetic is generally used for containment function?
   - a. Geotextile
   - b. Geomembrane
   - c. Geonet–geotextile geocomposite
   - d. None of the above

4. Properties of geosynthetics that are important for reinforcement application in a gravelly soil are:
   - a. High modulus
   - b. High tensile strength
   - c. High geosynthetic–soil interface shear strength
   - d. All of the above

5. Properties of a geotextile that are important for separation function are:
   - a. High tear strength
   - b. High burst strength
   - c. High tensile strength
   - d. All of the above

6. Which of the following statements on AOS of a geotextile is not true?
   - a. AOS is the approximate diameter of the largest particle size that would effectively pass through the geotextile.
   - b. At least 95 percent of the openings of the geotextile apparently have the diameter corresponding to AOS or smaller.
   - c. At least 5 percent of the openings of the geotextile apparently have the diameter corresponding to AOS or smaller.
   - d. AOS is dependent on the stress conditions on the geotextile.

7. If the O_{95} of a geotextile is 0.21 mm, then:
   - a. 95 percent of the pores in the geotextile are 0.21 mm or smaller.
   - b. 5 percent of the pores in the geotextile are 0.21 mm or smaller.
   - c. 100 percent of the pores in the geotextile are 0.21 mm or smaller.
   - d. None of the above.

8. The common unit of transmissivity is:
   - a. m/s
   - b. s^{-1}
   - c. m^3/s
   - d. m^2/s

9. The common unit of permittivity is:
   - a. m/s
   - b. s^{-1}
   - c. m^3/s
   - d. m^2/s

10. A multifilament woven geotextile generally has:
    - a. Lower tensile strength than a nonwoven geotextile.
    - b. Higher extension at failure than a nonwoven geotextile.
    - c. Lower extension at failure than a nonwoven geotextile.
    - d. None of the above.

11. A property, or properties, of a geosynthetic that is important for use as a barrier in a landfill is:

a. High thickness
b. Low cross-plane permeability
c. High tensile strength
d. All of the above

12. The ultimate tensile strength of a geogrid is 270 kN/m, and the combined reduction factor for reinforcement application is 3.0. The allowable tensile strength is:

a. 270 kN/m
b. 135 kN/m
c. 90 kN/m
d. 510 kN/m

13. A PET multifilament woven geotextile is being considered for use as a basal reinforcement for an embankment on a soft clay foundation, and you have been asked to determine its tensile strength and modulus for design. What test you will carry out? Describe how you will process the data to obtain the required tensile properties.

14. The following results were obtained from a series of geotextile–sandy fill interface direct shear tests conducted at different normal stresses.

Normal stress (kPa)	Shear strength (kPa)
50	46.1
100	77.7
150	108
200	141

Plot the Mohr-Coulomb type shear strength envelope and determine the adhesion and interface friction angle

15. Referring to the previous question, estimate the pullout capacity of the geotextile from the sandy fill under an average normal stress of 40 kPa. State all your assumptions and comment on the accuracy of the predicted pullout capacity.

16. What is the allowable tensile strength of a geosynthetic for designing a reinforced soil structure and how is it estimated?

17. You are required to select a geosynthetic for use as filter behind a retaining wall. What are the critical considerations? Outline how you will select the geosynthetic.

18. What are considerations for selecting a geosynthetic for separation function beneath the gravel base of an unpaved road to be constructed on soft clay subgrade?

19. Geogrids are commonly used as reinforcement. Discuss the reasons why they are not used for separation function.

20. List the geosynthetic properties that are related to the function of (a) filtration, (b) fluid barrier, (c) drainage.

Answers to questions:

1. c 2. b 3. b 4. d 5. d 6. c
7. a 8. d 9. b 10. c 11. d 12. c
14. Adhesion $= 15$ kPa, $\delta = 32°$.

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7

CHAPTER

The Pitch Lake, Trinidad is one of the largest natural asphalt deposits in the world. Anton_Ivanov/Shutterstock.com

Asphalt Cement and Hot Mix Asphalt Concrete

7.1 Introduction

Asphalt cement is the binding material in *asphalt concrete*, which is also referred simply as *asphalt* in the United States. Asphalt concrete is a mixture of aggregates and asphalt cement produced in a mixing plant. Asphalt concrete (AC) is also referred as hot mix asphalt (HMA) or HMA concrete. *Asphalt* is referred to as *bitumen* in Australia, Europe, and elsewhere. *Bitumen* is defined as "any of various tarlike mixtures of hydrocarbons derived from petroleum naturally or by distillation and used for road surfacing and roofing" in the Oxford Dictionary. In addition to road pavement and roofing, bituminous materials have found wider use these days in applications such as for damp proofing and waterproofing of floors in buildings, canal or reservoir linings, dam groutings, insulating boards, backed felts, and in various industrial products such as brake linings, insulating felts, battery boxes, and electrical insulating compounds.

Bituminous materials are classified as asphalts and tars. Naturally occurring bituminous materials have been found close to geological faults in underground crude oil deposits. For thousands of years, there had been occasional surface seepage of natural asphalt in the Middle East where extensive crude oil deposits are found. The inhabitants of these areas quickly realized the adhesive, waterproofing, and preservative nature of these materials and used them for various purposes.

Asphalt had been used in one form or another as a waterproofing and/or bonding agent, and the earliest recorded use was by the Sumerians, whose empire existed between 3500 BC to around 2000 BC (Read and Whiteoak, 2003). However, the best-known natural form of asphalt was the *lake asphalt* found as surface deposits in Trinidad, believed to have been discovered by Sir Walter Raleigh in 1595 and used as an excellent material for caulking (or waterproofing) the seams of ships. Although there are several small natural deposits of asphalt in Trinidad Island, the Pitch Lake or Trinidad Lake asphalt in the southern part is one of the largest in the world (approximately 35 hectares and 90 m deep, over 10 million tonnes).

Natural deposits of *rock asphalt*, that is, asphalt-impregnated rock, found in Europe and North America have been extracted from mines or quarries. In the United States, the vast bituminous sandstone and schist rock asphalt deposits found in Utah and Kentucky were mined until the early twentieth century. For example, the Sunnyside sandstone rock asphalt deposit in Utah is estimated to be 800 million tonnes with 8 to 13 percent asphalt concentration. It would be sufficient to construct a twenty-two-meter-wide road three times around the earth (Read and Whiteoak, 2003). It is worth noting that rock asphalt was used for waterproofing the surface course of the very first roads in Europe and North America, and a typical example is Union Square in New York that was surfaced using the rock asphalt from Switzerland in 1872.

Another form of natural asphalt is *gilsonite* found in Utah, which was first discovered in 1860 and subsequently used as a waterproofing agent for timber by Samuel Gilson in 1880. This asphalt is very hard and consequently difficult to mine. It is therefore, costly and less attractive for use in road pavement construction. However, it has found use in combination with other softer asphalts, for example, in bridge and roof waterproofing materials.

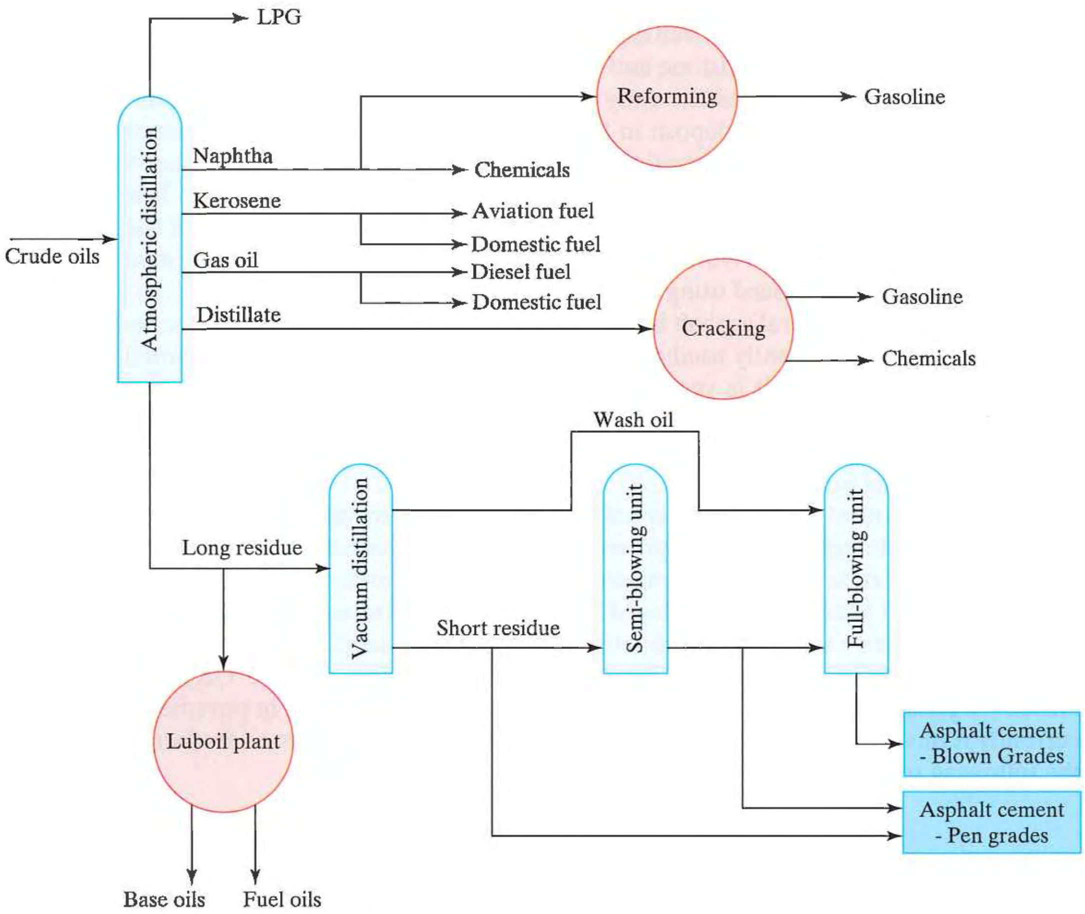
Although natural deposits of asphalt materials found in various places have been used for different purposes, at present asphalt for road construction is primarily sourced from crude oil. Major oil-producing areas in the world are North America (US, Canada, and Mexico), Russia, the Middle East (Saudi Arabia, Iraq, Iran, UAE, Kuwait, Qatar, and Oman), China, South America (Venezuela, Brazil, Columbia, Argentina, and Ecuador), Europe (European Union, Norway, and the UK), and Africa (Nigeria, Angola, Algeria, Egypt, and Libya). The asphalt used in pavements currently is mostly manufactured in crude oil refineries, and it is discussed further in the following sections.

7.2 Types of Asphalt Cements or Bituminous Materials

Asphalt cement for road construction is obtained predominantly from crude oil. Crude oil is a complex mixture of hydrocarbons predominantly paraffinic, naphthenic, or aromatic and physically varies from black to straw-colored viscous liquid. It is refined through a fractional distillation process. A schematic representation of it is shown in Figure 7.1. Asphalt cements are low-value products obtained from the residue after the high-value products such as gasoline, chemicals, aviation fuel, and diesel fuel have been extracted. The physical and chemical properties of crude oils differ widely, and the amount and quality of asphalt cement derived depends on the crude oil source as well as the distillation processes. For example, Nigerian crude oil typically yields only small amount of asphalt residuum.

Asphalt cement is a mixture of a very large number of chemical compounds, predominantly of hydrocarbon structure, with small amounts of sulphur, nitrogen, and oxygen. Asphalts typically contain 82–88 percent carbon, 8–11 percent hydrogen, 0–6 percent sulphur, 0–1.5 percent oxygen, and 0–1 percent nitrogen, and the precise composition varies depending on the crude oil source and the distillation/manufacturing process (Read and Whiteoak, 2003). It is resistant to most acids, alkalis, and salts.

Figure 7.1 Schematic representation of crude oil distillation process



(Based on Read, J. and Whiteoak, D. (2003). The Shell Bitumen Handbook. Fifth Edition. Thomas Telford Publishing.)

In general, asphalt cement is a viscous liquid at normal operating high temperature such as 150–160 °C (302–320 °F) but solid at normal air temperature. However, liquid asphalt cement products (for example, emulsified asphalts) have also been developed, which can be used at low temperature, but they do not produce the high-quality asphalt concrete mix preferred for the base and surface layers in pavement. The asphalt concrete for pavement construction is therefore commonly prepared at high temperature by mixing liquid asphalt (at high temperature) with hot aggregates, and it is referred to as *hot mix asphalt* (HMA). It is worth noting that HMA is spread and compacted while hot during pavement construction. Key desirable characteristics of asphalt include:

- Strongly adherent to most materials
- Water resistant and hence used for waterproofing and sealing
- Flexible and ductile
- Durable
- Low toxicity
- Economical as binder for sprayed seals and aggregate mixes (HMA) in road pavements

7.3 Common Asphalt Cement Products and Their Grading

Asphalt cement is also produced in different forms for use in pavement construction, such as *cutback asphalt* and *emulsified asphalt*. Cutback asphalt is basically a combination of asphalt cement and a petroleum solvent, such as kerosene or aviation turbine fuel, to temporarily reduce its viscosity. Emulsified asphalt is a liquid product in which asphalt cement is suspended in a finely divided condition in water by means of an emulsifying agent such as soap. Grades and designations of common emulsified asphalt products are discussed later in this chapter. A couple of other products, namely foamed asphalt and tar, are also used in certain pavement construction works, each of which will also be discussed in this section.

Grading: Asphalt cement products used in pavements have traditionally been graded in terms of two basic characteristics, namely *standard penetration* and *viscosity*. However, there are several grading methods used for asphalt, and the common methods used in pavements are *standard penetration, viscosity, aged residue*, and *performance*. Details concerning these grading methods are discussed below.

7.3.1 Standard Penetration Grading

The equipment for performing the standard penetration test is shown in Figure 7.2a. Typically, the penetration of a standard-sized needle under the weight of a 100 g mass into the asphalt cement sample, such as that shown in Figure 7.2b, at 25 °C (77 °F) over a time interval of 5 seconds (5 s) is measured as illustrated in Figure 7.2c. The measured penetration in units of 0.1 mm is reported as the *standard penetration* of the asphalt cement. However, alternate equivalent mass for the penetration load, penetration duration, and temperature of asphalt cement could also be used for the penetration test, and these details are presented, for example, in ASTM D5.

The common penetration-grade asphalts used in pavement construction are summarized in Table 7.1, and details regarding their properties are typically found in ASTM D946.

7.3.2 Viscosity Grading

The ratio between the shear stress (τ) and the rate of shear strain ($d\gamma/dt$) is defined as the viscosity (η). For a Newtonian fluid, the variation of shear stress with rate of shear strain is linear and hence a constant, where

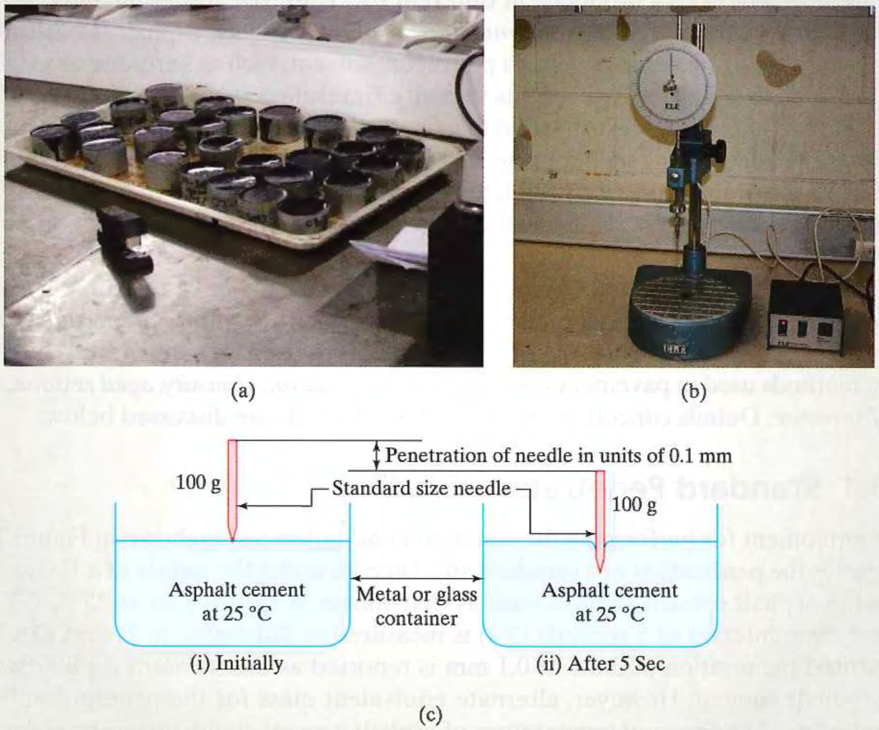
$$\tau = \eta \frac{d\gamma}{dt} \tag{7.1}$$

Viscosity indicates the resistance to flow of the asphalt cement when it is in fluid form and at a specific temperature. The SI unit of viscosity is Pa s or N.s/m^2. The *cgs* unit of viscosity is g/cm.s or dyne.s/cm^2, and it is called poise (P). It is worth noting that, 1 mPa s = 0.01 P and 1 Pa s = 10 P.

The kinematic viscosity is defined as the ratio between the viscosity and the density, both at the same temperature. The SI unit for kinematic viscosity is m^2/s, and the *cgs* unit alternative is cm^2/s, which is commonly referred to as "stoke" with the symbol "St."

$$10^{-6}\,\text{m}^2/\text{s} = 1\,\text{mm}^2/\text{s} = 10^{-2}\,\text{St} = 1\,\text{cSt}.$$

Figure 7.2 Standard penetration test: (a) samples of asphalt cement for penetration testing (b) standard penetration test equipment (c) schematic of standard penetration test



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Table 7.1 Penetration-grade asphalt cements used in pavement construction (ASTM D946)

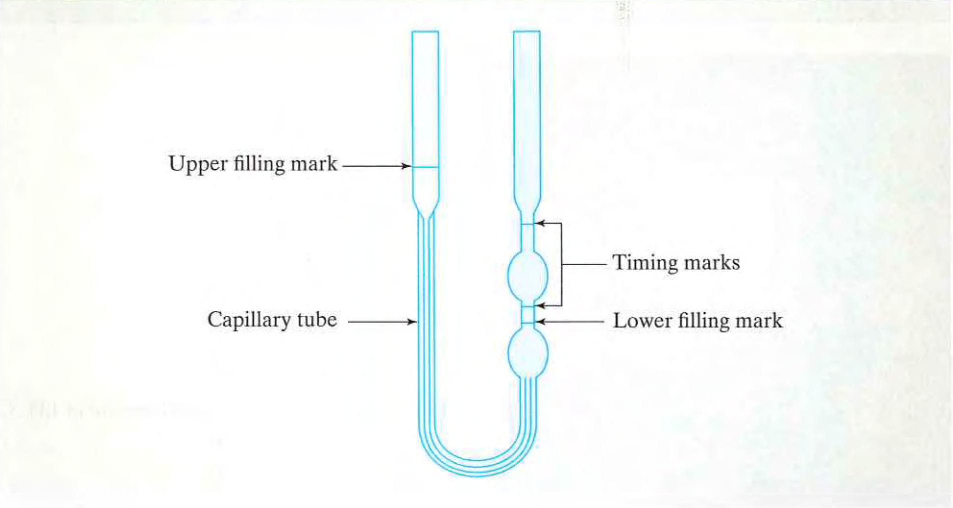
Penetration Grade	Minimum penetration at 25 °C (77 °F), 100 g, 5 s	Maximum penetration at 25 °C (77 °F), 100 g, 5 s
40–50	40	50
60–70	60	70
85–100	85	100
120–150	120	150
200–300	200	300

(Based on ASTM D946)

Different types of viscometers have been developed for measuring the viscosity or kinematic viscosity of various asphalt cement products, and the schematic of a typical capillary tube gravitational flow viscometer is shown in Figure 7.3. In this method, a certain quantity of asphalt cement is filled between the upper- and lower-level marks of the viscometer. The asphalt cement–filled viscometer is kept in a water tank that is maintained at the specified constant temperature (typically 60 °C or 140 °F), and the asphalt cement is allowed to flow through the capillary tube of standard dimensions

Schematic of a typical gravitational flow-type capillary viscometer

Figure 7.3



under gravity. The time taken for the asphalt cement to flow through from the lower to the upper mark of the viscometer is measured. The kinematic viscosity of asphalt cement is then calculated by multiplying the time t (s) by the viscometer calibration constant, $C\ (\mathrm{mm}^2/\mathrm{s}^2)$.

$$\text{Kinematic viscosity, mm}^2/\text{s} = Ct. \tag{7.2}$$

Further details regarding the viscometer, including the determination of the calibration constant, are found, for example, in ASTM D2170. Depending on the viscosity of the asphalt cement, an alternate vacuum-type capillary viscometer has also been developed for use, and its details are found in ASTM D2171.

The common viscosity grade asphalts used in pavement construction are summarized in Table 7.2, and details regarding their typical properties are found, for example, in ASTM D3381. The grade designation number (e.g., 5 in AC-5 grade) reflects one tenth of the mean value of the viscosity in Pa s, or alternatively, it is equal to 1/100 of the mean value of viscosity in poise. Given in the last column of Table 7.2 are the

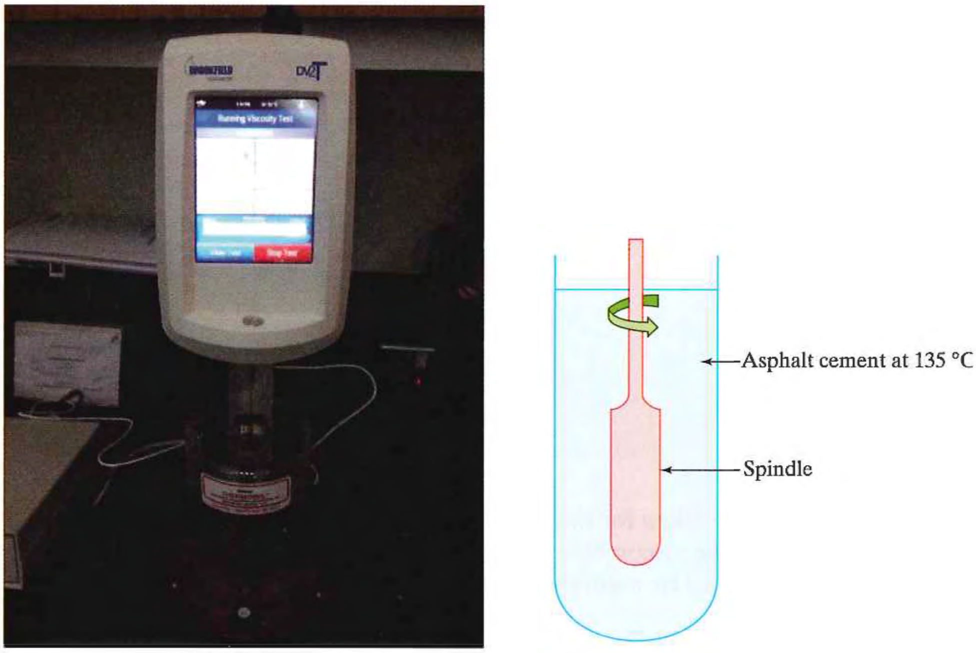
Viscosity grade asphalt cements used in pavement construction and their standard penetration (ASTM D3381)

Table 7.2

Viscosity grade asphalt cement	Viscosity at 60 °C (140 °F) Pa s	Minimum penetration according to ASTM D3381 25 °C (77 °F), 100 g, 5 s
AC-2.5	25 ± 5	200
AC-5	50 ± 10	120
AC-10	100 ± 20	70
AC-20	200 ± 40	40
AC-30	300 ± 60	30
AC-40	400 ± 80	20

(Based on ASTM D3381)

Figure 7.4 Brookfield rotational viscometer apparatus: (a) photo (b) schematic of the spindle



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

typical minimum standard penetrations required in accordance to ASTM D3381 for the viscosity grade asphalt cements, which gives an indication of the approximate relationship between the penetration and viscosity of asphalt cements used in pavement construction.

The viscosity of asphalt cement at temperatures ranging from 38 to 260 °C (100 to 500 °F) could be determined using a (Brookfield) rotational viscometer (ASTM D4402). The Brookfield apparatus shown in Figure 7.4 consists of a rotational coaxial cylinder viscometer and a temperature-controlled thermal chamber.

A known volume of the asphalt cement sample is poured in the chamber at 135 °C, and the spindle is submerged in the sample. Sample and the spindle are maintained at the test temperature, and the spindle is rotated at a specified rate. The torque required to rotate the spindle in the asphalt is measured, and the viscosity is evaluated.

7.3.3 Aged Residue Grading

A recently developed grading requirement for asphalt cement in pavement construction is based on the properties of the aged residue from the rolling thin film oven test, summarized in Table 7.3. The grade designation number (for example, 1000 in AR-1000) reflects ten times the mean value of the viscosity in Pa s, or alternatively it is equal to the mean value of viscosity in poise. Details regarding the typical properties of the different grades are found, for example, in ASTM D3381. The aged residue from the rolling thin film oven test (ASTM D2872) approximately simulates the short-term aging of the asphalt cement by the loss of volatiles due to the heating (to, say, about 163 °C [325 °F]) during the manufacture of hot mix asphalt and placing and compacting in the field.

Grading based on the viscosity of aged residue (ASTM D3381)

Table 7.3

Viscosity grade based on the testing of residue	Viscosity at 60 °C (140 °F) Pa s
AR-1000	100 ± 25
AR-2000	200 ± 50
AR-4000	400 ± 100
AR-8000	800 ± 200
AR-16000	1600 ± 400

(Based on ASTM D3381)

7.3.4 Performance Grading

Asphalt cement is the binder in HMA for its use in pavements, and its performance in the field will depend on pavement temperature, which varies during the service life of the pavement. Performance grading (PG) designation of asphalt cement binder developed more recently is based on average seven-day maximum pavement design temperature and minimum pavement design temperature. The available performance grade asphalt cements are summarized in Table 7.4 (ASTM D6373). An example would be, for a pavement that has a design high temperature of 64 °C and design low temperature of −28 °C, the performance grade asphalt cement PG 64-28 is selected.

The performance-graded asphalt binder specifications are based on the dynamic shear (ASTM D7175), creep stiffness (ASTM D6648), and direct tension failure strain (ASTM D6723) properties of the asphalt binder. Details of the performance graded asphalt specifications could be found in ASTM D6723.

7.3.5 Other Asphalt Products

Cutback asphalt: Asphalt cement is combined with a controlled amount of relatively volatile petroleum solvent such as kerosene or aviation turbine fuel to temporarily reduce its viscosity; the product is called cutback asphalt. The addition of solvent is to enable the cutback asphalt to penetrate the pavement by properly wetting and adhering to the aggregates easily or to enable spraying (e.g., low-temperature

Performance grade asphalt cement designations

Table 7.4

		High temperature (°C)	Low temperature (°C)						
Performance Grade	PG	46	–46	–40	–34				
	PG	52	–46	–40	–34	–28	–20	–16	–10
	PG	58		–40	–34	–28	–20	–16	
	PG	64		–40	–34	–28	–20	–16	–10
	PG	70		–40	–34	–28	–20	–16	–10
	PG	76			–34	–28	–20	–16	–10
	PG	82			–34	–28	–20	–16	–10

use). Most of the solvent will evaporate in a relatively short time (several days to months), and the asphalt cement residue remains on the surface to which it was applied. The solvent added is generally volatile and therefore likely to be hazardous. Moreover, evaporation of the solvent into the environment could be a concern, and restrictions on the use of cutback asphalt exist in many places due to environmental regulations.

Emulsified asphalt: It is a liquid product in which the asphalt cement is suspended in a finely divided droplet condition in water by means of an emulsifying agent such as soap. The emulsifying agent enables the development of a negative or positive charge to the asphalt cement so that the particles do not coalesce but rather are finely divided. Emulsions in effect reduce the viscosity of the asphalt cement, thereby enabling low temperature use.

Depending on the type of emulsifying agent used, they are classified as anionic emulsions (i.e., asphalt cement droplets are negatively charged at the discontinuous phase) or cationic emulsions (i.e., asphalt cement globules are positively charged at the discontinuous phase). When the asphalt cement globules in the emulsion start adhering to the pavement material (e.g., aggregate or an existing pavement layer) onto which the emulsion is applied, the color of the applied asphalt emulsion changes typically from dark brown to black and the emulsion is said to have broken. As time passes, the water in the emulsion starts to evaporate, and the material begins to behave essentially like pure asphalt cement. When all the water from the asphalt emulsion evaporates, it is said to have set. The time it takes to break and set are dependent on the type of emulsion used, its application rate, pavement material on to which it is applied, and the environmental conditions (e.g., temperature, humidity, etc.). Depending on the time for setting, emulsified asphalts are classified as slow setting, medium setting, or rapid setting. More recently, quick-setting emulsions are also being manufactured particularly for slurry seals (e.g., Quick Set Slurry Seal systems). Standard test methods and practices for emulsified asphalt are found in ASTM D244, and it is worth noting that the viscosity of emulsified asphalt is commonly determined by using a rotational paddle viscometer (ASTM D7226) or a Saybolt Furol viscometer (ASTM D7496).

In essence, emulsified asphalts are classified by type (i.e., slow, medium, or rapid setting) and the grade within the type as indicated in Table 7.5. Rapid-setting asphalt emulsions are graded according to viscosity, whereas medium- and slow-setting asphalts are graded in accordance to the characteristic of the residual asphalt.

Selection of the type and grade of asphalt emulsion is dependent on the use, for example, whether it is for asphalt concrete mix or surface treatment. For asphalt concrete mix, the grading and mineral composition of the aggregate and environmental conditions at the time of construction also need to be considered for selecting the asphalt emulsion binder. Slow breaking is generally required when time is needed for mixing, transporting, and placing the asphalt concrete mix. For surface treatments and seals, a rapid-setting asphalt emulsion is often sought. Details concerning the selection and use of emulsified asphalts are found in ASTM D3628.

Foamed asphalt: Foamed asphalt is a mixture of air, water, and hot asphalt cement. Injecting a small quantity of cold water into hot asphalt produces an instantaneous expansion of the asphalt, up to fifteen times its original volume (Austroads, 2006a). The concept of manufacturing foamed asphalt is illustrated in Figure 7.5. The increased surface area improves the asphalt's viscosity and enables mixing with damp and cold aggregates for making asphalt concrete mix.

Emulsified asphalt grades

Table 7.5

	Grade	Setting speed	Viscosity—Rotational paddle		Residue by distillation (min), %	Penetration on residue 25 °C, 100 g, 5 s
			at 25 °C mPa s	at 50 °C mPa s		
Anionic emulsions	RS-1	Rapid	45–220		55	100–200
	RS-2	Rapid		165–880	63	100–200
	HFRS-2	Rapid		165–880	63	100–200
	MS-1	Medium	45–220		55	100–200
	MS-2	Medium	220 (min)		65	100–200
	MS-2h	Medium	220 (min)		65	40–90
	HFMS-1	Medium	45–220		55	100–200
	HFMS-2	Medium	220 (min)		65	40–90
	HFMS-2h	Medium	220 (min)		65	40–90
	HFMS-2s	Medium	110 (min)		65	40–90
	SS-1	Slow	45–220		57	40–90
	SS-1h	Slow	45–220		57	40–90
	QS-1h	Quick	45–220		57	40–90
Cationic emulsions	CRS-1	Rapid		20–100	60	100–250
	CRS-2	Rapid		100–400	65	100–250
	CMS-2	Medium		50–450	65	100–250
	CMS-2h	Medium		50–450	65	40–90
	CSS-1	Slow	20–100		57	100–250
	CSS-1h	Slow	20–100		57	40–90
	CQS-1h	Quick	20–100		57	40–90

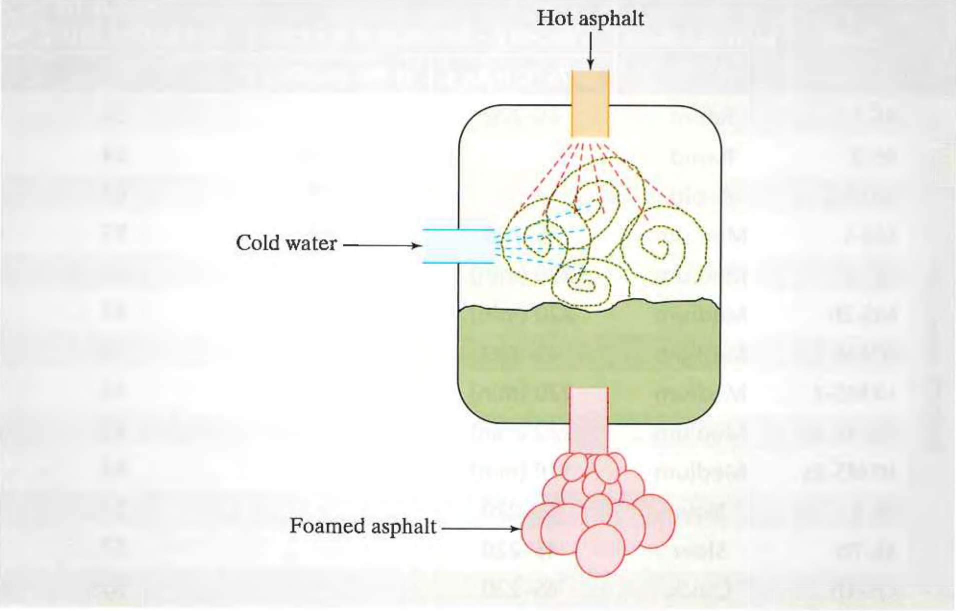
(Based on ASTM D977 and ASTM D2397)

Tar: A black material of hydrocarbons and free carbon obtained by carbonation or destructive distillation in the absence of air of natural organic materials such as coal and wood. There are three main types of tar:

> *Coke oven tar:* A byproduct in steel production during the process of turning coal into coke; the process is through destructive distillation.
> *Coal tar:* A byproduct of coal gas production, again from a process of destructive distillation of coal.
> *Petroleum tar:* Obtained by refining crude oil by thermal cracking at high temperature.

Tar has excellent adhesive properties but is more temperature susceptible than asphalt cement. Also, it is less durable than asphalt cement and therefore more suitable for priming work. However, some tars are carcinogenic. The use of tars in pavements has dramatically decreased over the past few decades, and it will not be discussed any further in this chapter.

Figure 7.5 Schematic of the manufacture of foamed asphalt



(Based on Austroads (2006). Guide to Pavement Technology - Part 4D: Stabilised materials. Austroads, Sydney, Australia. Austroads Publication No: AGPT04D/06.)

7.4 Typical Uses of Asphaltic Materials

Asphalt materials are used widely in civil engineering projects, such as in the construction of roads, for waterproofing the floors in buildings, and lining canals and reservoirs. However, its main use is in the construction of roads, such as the base and surface courses of roads using hot mix asphalt (or asphalt concrete). Typical hot mix asphalt (HMA) in a loose state is shown in Figure 7.6. Some typical road construction activities, involving HMA products, are shown in Figures 7.7 to 7.11 below.

7.5 Properties and Testing of Asphalt Cement

As discussed earlier, asphalt cement is a viscous liquid at high operating temperature but solid at normal air temperature. The consistency, rate of curing, and durability are important characteristics of asphalt cements, and they are assessed through different tests, which are discussed below. Some engineering properties of asphalt cement obtained from tests such as direct tension and bending beam rheometer will also be examined.

The temperature of a pavement structure usually changes with the time of the day and seasonal weather pattern. Therefore, the variation of consistency and other properties with temperature also need to be considered when designing asphalt concrete mix and treatment measures involving asphalt materials.

HMA in loose state

Figure 7.6

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Placement of HMA using a tracked paver for the surface course of a residential road

Figure 7.7

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 7.8 Dynamic roller compaction of the HMA surface layer of a residential road

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Figure 7.9 Multiwheel roller compaction of the HMA surface layer of a residential road

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

7.5.1 Consistency and Rate of Curing of Asphalt Cement

In the liquid state, the consistency of asphalt cements is typically assessed, using viscosity or kinematic viscosity. In semisolid or solid states, the penetration test is commonly used. However, the float test and sometimes the ring-and-ball softening point test is also used for certain types of asphalt (for example, blown asphalt).

Applying an HMA wearing course layer for an arterial road pavement Figure 7.10

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Roller compaction of the HMA wearing course of an arterial road pavement Figure 7.11

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Blown asphalt is oxidized asphalt, typically used for sealing saw cuts and joints where minimum lateral opening and movement in the joint are expected. Its consistency over a wide temperature range prevents bleeding in high-temperature applications.

Viscosity

Unlike the penetration and other index properties, viscosity of the asphalt is an engineering property. However, viscosity of asphalt changes with temperature, and it is often determined at different temperatures, say, for example, at 60 °C (140 °F) and 135 °C (275 °F). Determination of viscosity, using a gravitational flow type capillary viscometer (ASTM D2170) and a (Brookfield) rotational viscometer (ASTM D4402) were discussed previously in Section 7.2.2.

Penetration test

The penetration test gives an empirical measurement of the consistency of an asphalt material in terms of the distance a standard needle sinks into that material under a prescribed load and time. This test is usually carried out in a temperature-controlled water bath, and the standard penetration test protocols were discussed previously in Section 7.2.1.

Float test

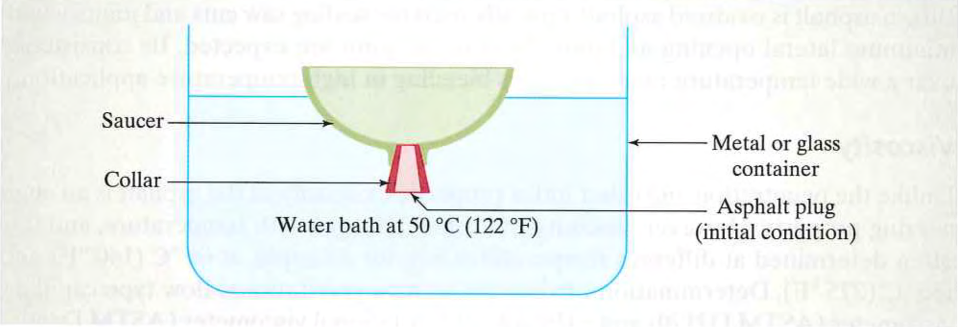
The testing equipment consists of a standard-sized brass collar, which is filled with a sample of the asphalt to be tested and then attached to the bottom of the aluminum saucer. The asphalt plug is initially chilled to 5 °C (41 °F) by immersing it in ice water. At the start of the test, the collar attached to the saucer is placed in a water bath to float. A schematic of a float test arrangement is illustrated in Figure 7.12. The temperature of the water in the bath is maintained constant at 50 °C (122 °F). As time passes, the warm water gradually softens the asphalt plug, and the head gradually forces the water through the plug into the aluminum saucer. The time, in seconds, that it takes from the instant the collar is placed in the water bath to the time when the water forces its way through the asphalt plug is the float test value. The float test is used to determine the consistency of semisolid asphalt or bituminous materials that are more viscous (typically, penetration >300) because they cannot be tested conveniently either by the Saybolt Furol viscosity test (ASTM D7496) or the penetration test (Garber and Hoel, 1997).

Ring-and-ball Softening Point Test

The ring-and-ball softening point test is used to measure the susceptibility of asphalt to temperature changes by determining the temperature at which the material is adequately softened to allow a standard ball to sink through it. The testing apparatus consists primarily of a brass ring of 15.9 mm (5/8 in.) inside diameter and 6.4 mm (1/4 in.) high, a steel ball 9.5 mm (3/8 in.) in diameter, and a water or glycerine bath. The test is conducted by first placing a sample of the asphalt cement to be tested in the brass ring, which is cooled and immersed in the water or glycerine bath that is maintained at a temperature of 5 °C (41 °F).

The testing equipment for a two-ring setup is shown in Figure 7.13 in which the two rings are supported on a ring-holder with a tray at 25 mm (1 in.) below the ring.

Figure 7.12 Schematic of float test setup



(Based on Garber, N.J. and Hoel, L.A. (1997). Traffic and Highway Engineering, Second Edition. PWS Publishing Co. Ltd.)

The steel balls are placed precisely at the center of each ring, using a special device. The glycerine bath containing the asphalt sample in the ring supporting the ball at the center is heated, and the temperature of the bath is monitored. The heating causes the asphalt to soften and the ball sinks into the asphalt. When the asphalt gets sufficiently softened, the ball sinks through the asphalt and drops onto the tray beneath. The temperature of the glycerine bath at which the ball sinks and touches the bottom tray is recorded as the *softening point*. Further details concerning the ring-and-ball softening point test can be found in ASTM D36.

It is worth noting that for many types of asphalt, the softening point = T_{800PEN} and the viscosity of majority of asphalts at softening point is 1200 Pa s (Read and Whiteoak, 2003), where T_{800PEN} = temperature at which the penetration (100 g mass in 5 s) of the asphalt is 0.1 mm (800 units).

Rate of Curing

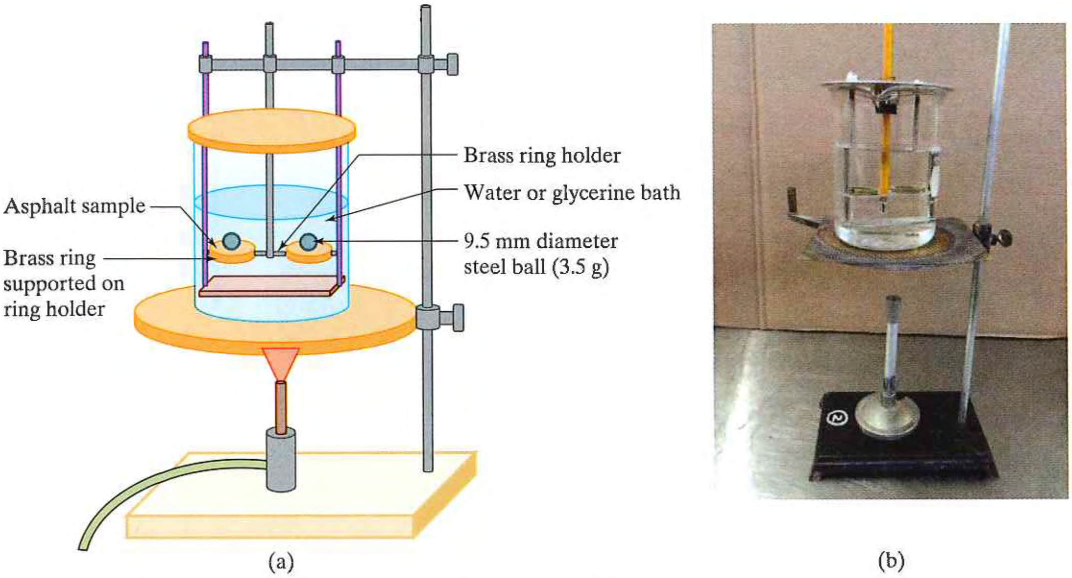
This is the process through which an asphaltic material increases its consistency as it loses solvent by evaporation. This is similar to concrete curing with water, steam, and so forth.

7.5.2 Durability and Other Properties of Asphalt Materials

The asphaltic materials used in pavements are subjected to temperature changes (for example, freezing and thawing) and other weather conditions over their service life. As time passes, oxidation, volatilization, and age hardening takes place in the

Ring-and-ball softening point testing equipment (a) Schematic of test set-up (b) Photo of testing equipment

Figure 7.13



(a)

(b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

asphaltic material, which could lead to loss of plasticity, cracking, changes in surface roughness, and eventual pavement failure. *Oxidation* refers to the asphalt material reacting with the oxygen in the air and the material getting hardened by it. This reaction depends on the temperature and leads to loss of plastic characteristics of the asphalt material. *Volatilization* refers to evaporation of lighter hydrocarbons over time; this is dependent on the temperature and exposed surface area of the pavement. Again, it leads to loss of plastic characteristics of the asphalt material. *Age hardening* refers to a change in molecular structure, say, for example, due to heating and cooling cycles, over time.

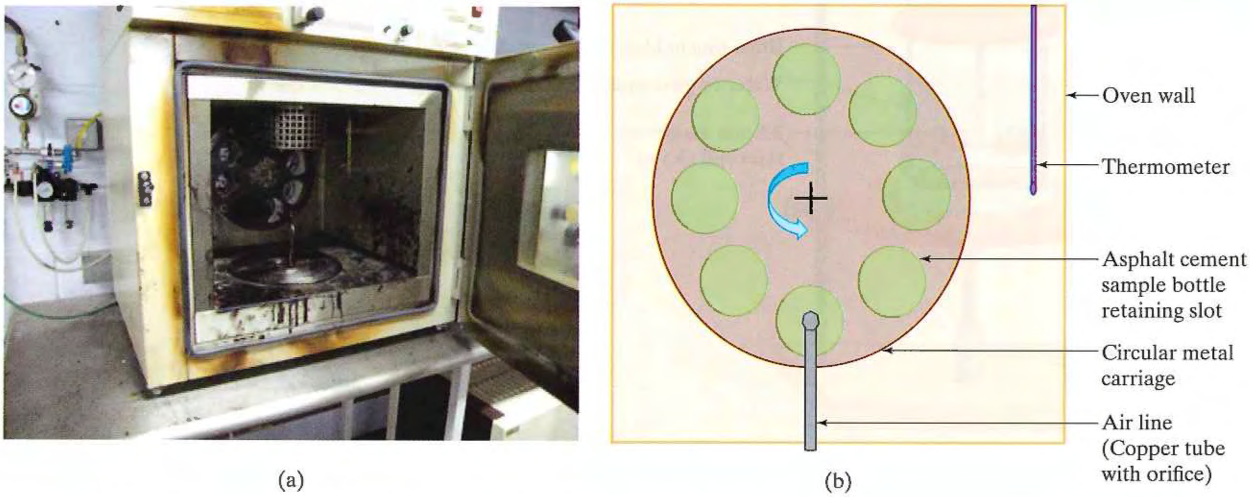
A method used for simulating the changes in the properties of asphalt cement due to short-term aging during the hot mix asphalt (asphalt concrete) manufacture is through the rolling thin film oven (RTFO) apparatus. In the RTFO method, a 35 g sample of asphalt cement is poured in a special transparent heat-resistant glass bottle. The sample bottle is then placed in the rotating shelf of the RTFO such as that shown in Figure 7.14 and rotated at 15 rev/min for 85 minutes with the air flowing and the asphalt cement heated to 163 °C (325 °F). The asphalt binder material is then tested accordingly for consistency and other properties to assess the effects of aging. Further details of the RTFO test protocols can be found, for example, in ASTM D2872.

There are various other tests for asphalt cement such as the ductility test, distillation test for emulsions and cutbacks, specific gravity test, solubility test, flash-point test, loss-on-heating test, water content test, and others. Some of the commonly performed tests in relation to pavement design are discussed further.

Ductility Test

The ductility test is used mainly for solid or semi-solid asphalt, which is initially heated gently and poured into the mold to prepare the standard-sized sample as that shown in Figure 7.15a. The ductility of asphalt cement is measured by stretching the standard-sized sample in a constant temperature (25 °C, 77 °F) water bath at a specified rate as shown in Figure 7.15b (ASTM D113, AASHTO T51). The distance

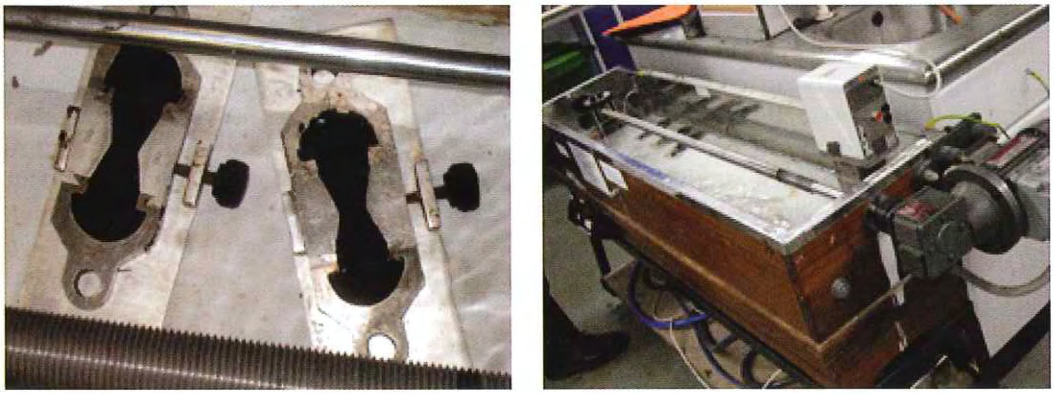
Figure 7.14 Rolling thin film oven (RTFO) test apparatus: (a) photo of apparatus (b) schematic of rotating shelf in RTFO



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Ductility testing apparatus: (a) close-up view of sample (b) experimental setup | Figure 7.15



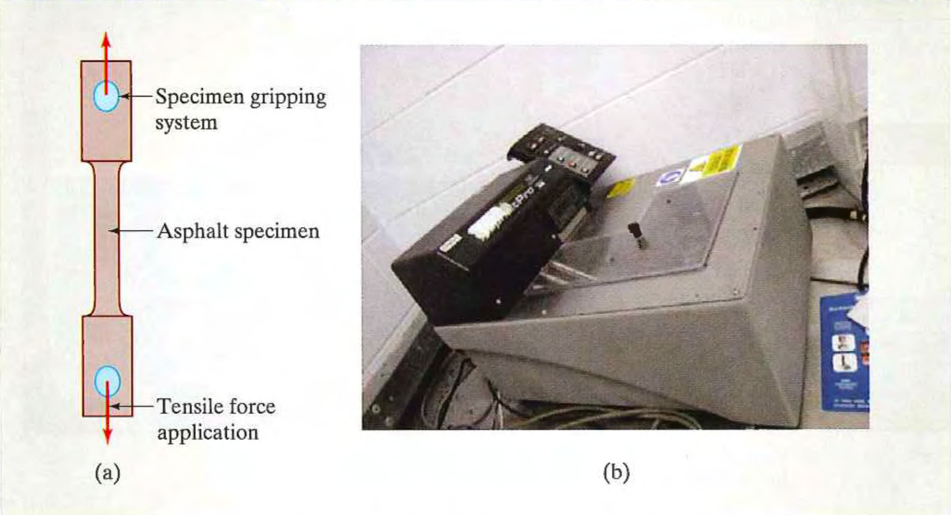
(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

through which the sample has stretched at breaking, in centimeters, is reported as the ductility. The ductility is an important characteristic that indicates the extent to which the asphalt material can be deformed before breaking.

Direct Tension Test

The failure strain and failure stress of an asphalt cement specimen when pulled at 1.0 mm/min elongation rate is determined in the direct tension test. The schematic of the specimen and the testing setup are shown in Figure 7.16. The test sample is prepared by pouring asphalt binder in a standard mold and tested in a special tensile testing apparatus in which the tensile force and elongation are monitored. The maximum tensile force indicates failure, and the corresponding stress and strain in the asphalt binder are calculated and reported as the failure stress and strain,

Direct tension testing of asphalt: (a) schematic of testing setup (b) photo of testing equipment | Figure 7.16



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

respectively. Further details of the direct tension test method can be found, for example, in ASTM D6723.

Bending Beam Rheometer (BBR) Test

A prismatic beam specimen (with $b \times h$ cross section) is prepared by pouring hot asphalt binder in a standard mold and allowing it to cool. The beam is then simply supported on special noncorrosive (e.g., stainless steel) supports in a controlled-temperature bath of fluid such as ethanol, methanol, or similar fluid and subjected to a constant mid-span test load P (= 980 ± 50 mN) for 240 seconds. The schematic of the specimen and the testing setup is illustrated in Figure 7.17a, and a photo of a BBR is shown in Figure 7.17b. The variation of mid-span deflection (δ) of the beam with time (t) is monitored using a computerized data acquisition system.

The mid-span deflection (δ) readings of the asphalt cement beam at 8, 15, 30, 60, 120 and 240 seconds are used for calculating the stiffness and other characteristics at different times as follows:

From elastic theory,

$$\text{Elastic flexural modulus of a prismatic beam, } E = \frac{\mathrm{P}L^3}{4b\,h^3\delta} \tag{7.3}$$

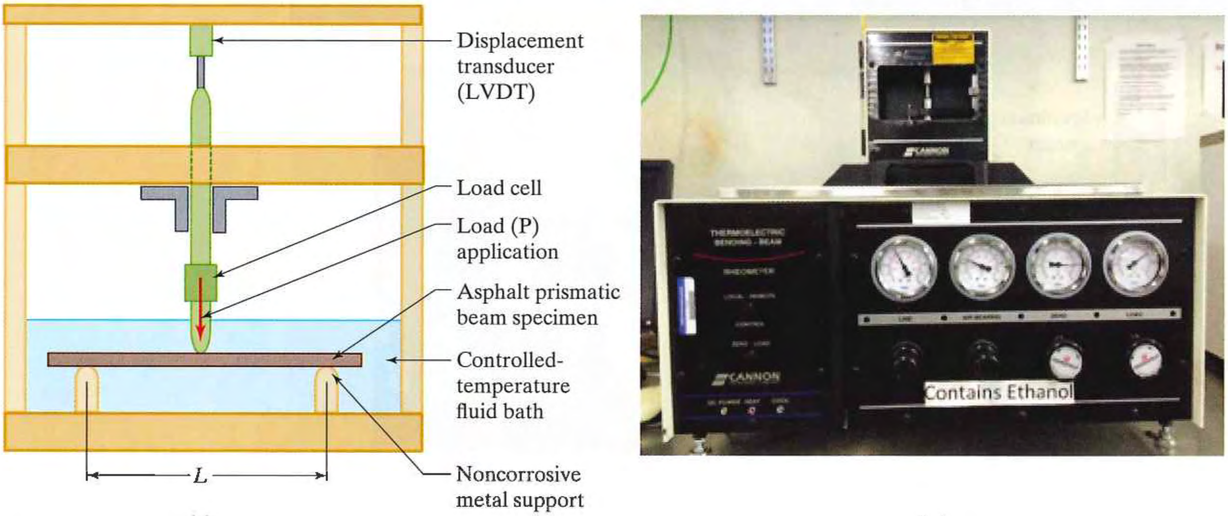
$$\text{Maximum bending stress, } \sigma = \frac{3\,\mathrm{P}L}{2\,bh^2}\ \text{(MPa)} \tag{7.4}$$

$$\text{Maximum bending strain, } \varepsilon = \frac{6\,\delta h}{L^2}\ \text{(mm/mm)} \tag{7.5}$$

where

P = applied mid-span load (N)
L = span length of specimen (mm)

Figure 7.17 BBR testing of asphalt: (a) schematic of test setup (b) photo of testing equipment



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

b = cross-sectional width of specimen (mm)
h = cross-sectional height of specimen (mm)
δ = mid-span deflection (mm)

The flexural-creep stiffness of asphalt binder at time t is determined, using the following equation, according to the linear viscoelastic correspondence principle:

$$\text{Flexural creep stiffness at time } t,\ s_m(t) = \frac{PL^3}{4bh^3\ \delta(t)}\ (\text{MPa}) \qquad \textbf{(7.6)}$$

where $\delta(t)$ = deflection of the beam at time t.

Further details regarding the BBR test method and the determination of flexural creep stiffness of asphalt cement can be found, for example, in ASTM D6648. It is worth noting that thermal cracking in HMA pavements is related to the flexural creep stiffness and slope of the variation of $\log(s_m(t))$ against $\log(t)$ of the asphalt cement used in the HMA.

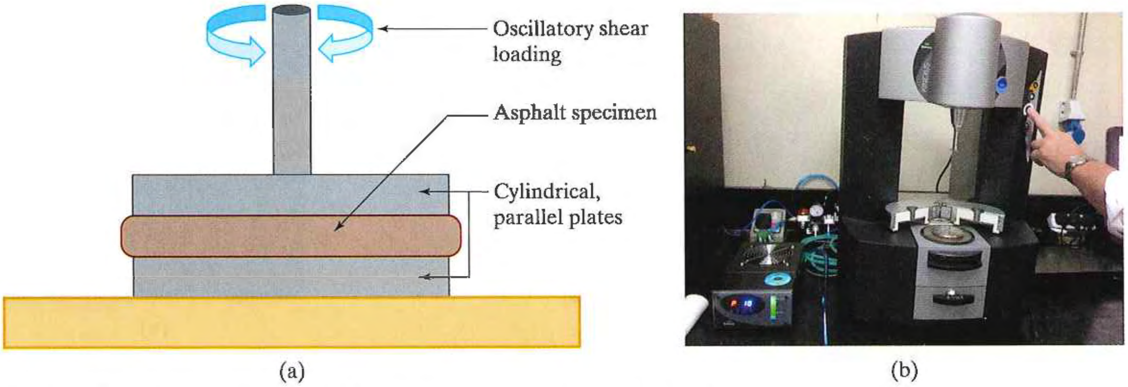
Dynamic Shear Rheometer (DSR) Test

Oscillatory-type dynamic shear loading is applied to the asphalt specimen sandwiched in between two concentric, cylindrical, parallel, metal plates as schematically shown in Figure 7.18a. A photo of a DSR is shown in Figure 7.18b. Depending on whether a stress- or strain-controlled test is being performed, shear stress or shear strain is applied to one plate with respect to the other, varying in a sinusoidal pattern with time at 10.0 ± 0.1 rad/s frequency. This test is performed in a temperature-controlled environmental chamber. The frequency, deflection angle, and torque are measured during the dynamic shear loading, in addition to the temperature.

The complex shear modulus and phase angle of the asphalt binder are determined from the DSR test and are typically used in Performance Grade Asphalt specification. Further details regarding the test method and the determination of complex shear modulus and phase angle of the asphalt binder using DSR can be found, for example, in ASTM D7175.

DSR testing of asphalt cement: (a) schematic of test setup (b) photo of test equipment

Figure 7.18



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

7.6 Overview of Pavements

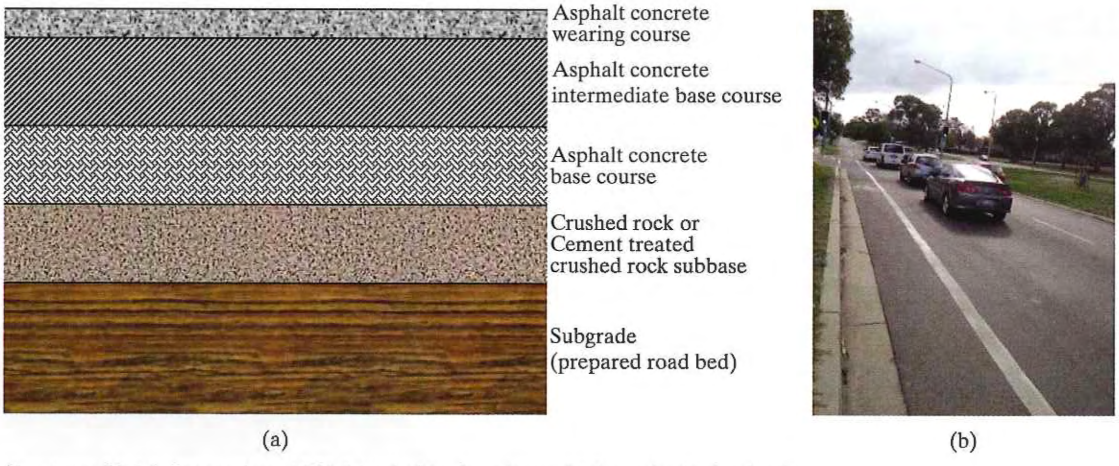
Pavement is a civil engineering structure built to carry traffic loads. The design procedure for pavement involves:

- Estimating the traffic loads the pavement must carry;
- Calculating the stresses induced within the pavement structure by the design traffic loads;
- Selecting the materials to build the pavement structure and determining their strength and stiffness properties; and
- Engineering the use of the materials in the pavement structure in such a way that the stresses induced by the loads are less than the strength of the materials.

There are two types of pavements, namely rigid and flexible pavements. Rigid pavements are primarily constructed of either a reinforced or plain concrete base with or without an asphalt concrete wearing course. A flexible pavement structure is a layered system in which better materials are used for the top layers where stress intensity is high and inferior materials at the bottom where the stress intensity is low. The upper structural layers are commonly referred as the base courses, which are underlain by sub-base layers followed by the prepared natural road-bed material, referred as subgrade. Schematic of the cross section of a typical urban road flexible pavement structure is shown in Figure 7.19a. A photo of a typical urban road flexible pavement is shown in Figure 7.19b.

Different types of asphalt concrete materials are generally used for constructing the upper layers of the flexible pavement, for example, the top three layers shown in Figure 7.19a, and such pavements are also referred to as *sealed road pavements*. For certain roads that carry a low volume of traffic, the top asphalt concrete layers may be absent, which are built entirely with unbound granular materials, and such pavements are referred as unpaved or unsealed or unbound granular roads. A thin layer of bituminous material could be sprayed over the unpaved road, commonly referred as surface sealing.

Figure 7.19 An urban road flexible pavement: (a) schematic of typical layers in the pavement (b) photo of an urban-road flexible pavement



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

The surface sealing mentioned above or the use of a thin asphalt concrete upper layer in a pavement is generally referred as bituminous surfacing. The purposes of providing bituminous surfacing in a road are to:

- Provide skid resistance in all types of weather conditions
- Eliminate dust production during the ride
- Maintain good riding quality to ensure a comfortable drive
- Protect the pavement underlayer from wear
- Protect the pavement underlayer from the effects of moisture

A bituminous surfacing treatment is divided into two types as sprayed work and asphalt concrete work.

Sprayed work: Hot bituminous material is sprayed onto a prepared pavement and then mostly covered with a thin layer of aggregate. There are several types of sprayed work, and details regarding the most common types are discussed in the next section.

Asphalt concrete work: Hot mix asphalt (HMA), also referred as asphalt concrete (AC), is used as the surface course and often as part of the base of the pavement. Properties of HMA are dependent on the mix, and details regarding the most common types are discussed in the next section.

7.7 Bituminous Surfacing Treatments and Sealing in Pavements

Sprayed work in pavements could be for initial treatment in new pavements, such as the compacted unbound granular base of a new pavement, or resealing the asphalt concrete (or bituminous) surfaces of existing pavements. The sprayed works in new pavement construction generally involve priming and sealing or sealing or primer sealing. However, various types of sealing works are carried out in pavement maintenance works, and some of the common types will also be examined in this section.

Priming: Spraying of suitable viscosity bituminous material (i.e., primer) over the road surface is to prepare it for subsequent application of a high-standard bituminous surfacing to provide a strong bond between the bituminous surfacing and the underneath pavement structure material.

Cutback asphalt or asphalt emulsion is commonly used as the primer. Also, primer could be applied onto a previously primed surface or to an existing asphalt/bituminous surface, to assist in the bonding with the next layer of the pavement. Application of primer on the granular base prior to the placement of HMA during the construction of a residential road is shown in Figure 7.20.

Sealing: Application of a thin layer of bituminous binder followed by a thin covering layer of aggregate is referred as sealing. This is mostly done after priming, but priming could be omitted in some circumstances. This provides a strong bond between the covering layer of aggregate and the underlying pavement as well as a waterproof seal.

Materials commonly used as binders for sealing are asphalt cement, fluxed asphalt, cutback asphalt, asphalt emulsion, and foamed asphalt. Variations of the sealing method are employed for different pavement construction and maintenance works, and some of the common types are discussed below.

Figure 7.20 View of typical primer application operation: (a) hand application of primer for a bus-stop area (b) priming the road using a tanker


(a) (b)

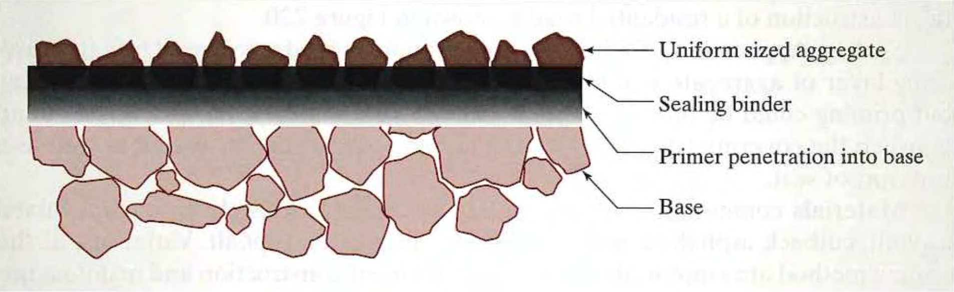
(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

Primer sealing: This is similar to sealing but a primer binder is used. A schematic of a primer seal is shown in Figure 7.21. Primer sealing is generally used as an interim (temporary) treatment prior to the application of a permanent bituminous surfacing which allows traffic to use the pavement for about twelve months.

Primer sealing allows for rectification of deficiencies and irregularities of the pavement, and it functions as both primer and seal (for the limited period). Cutback asphalt, asphalt emulsion, and tar are the materials commonly used as a primer binder.

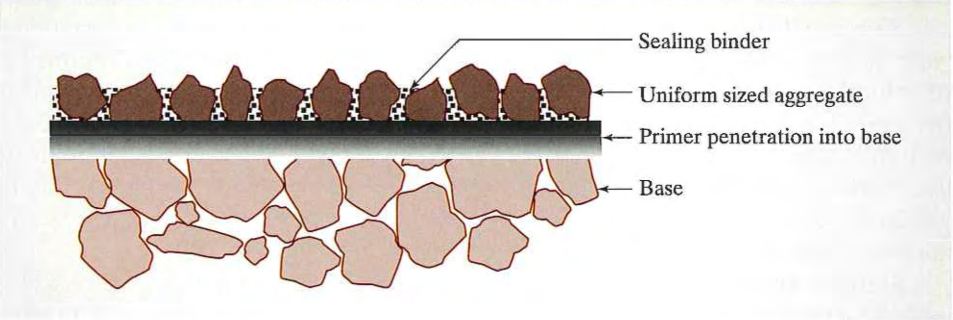
There may be more than one application of binder or binder-aggregate seal. A typical example of a subsequent sealing binder application over primer sealing, referred to as a single-single seal, is illustrated in Figure 7.22. Moreover, geotextile reinforcement could also be incorporated in the sealing work to minimize reflection cracking. A typical illustration of geotextile reinforced seal in the restoration or maintenance of a cracked old pavement is shown in Figure 7.23. Initially, binder is applied over the cracked pavement, which is followed by the laying of geotextile and a subsequent second application of the binder and the spreading of small-size uniform aggregate.

Figure 7.21 Primer seal



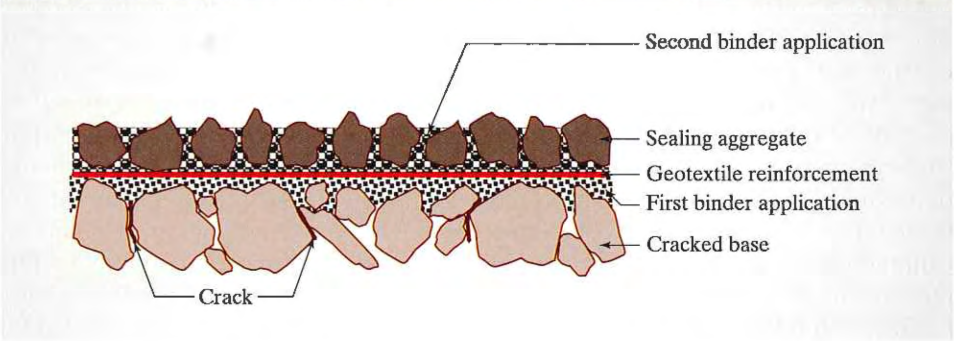
(Based on Austroads (2006). Update of the Austroads Sprayed Seal Design Method. Austroads Technical Report, by A. Alderson, AP-T68/06, Austroads, Sydney, Australia.)

Single-single seal **Figure 7.22**



(Based on Austroads (2006). Update of the Austroads Sprayed Seal Design Method. Austroads Technical Report, by A. Alderson, AP-T68/06, Austroads, Sydney, Australia.)

Geotextile reinforced seal **Figure 7.23**



(Based on Austroads (2006). Update of the Austroads Sprayed Seal Design Method. Austroads Technical Report, by A. Alderson, AP-T68/06, Austroads, Sydney, Australia.)

Chip sealing: This is a type of sealing where heated liquid asphalt is sprayed over the road surface and covered with a layer of small uniform-sized aggregate (i.e., chips). The chips are then compacted with rollers to orient the particles for better sticking with the asphalt, and the excess aggregate is often swept away. This is commonly used as a pavement maintenance operation for prolonging the life of a pavement before the need for an overlay of HMA.

For a sealing job, in general, the choice of aggregate size, binder type, and the design of aggregate and asphalt application rates are important factors. The purposes of sealing are:

- Provide a durable, safe, and comfortable riding surface
- Minimize the rate of pavement wear and maintenance cost
- Protect the base from the effects of traffic and the environment by preventing moisture ingress into the pavement structure
- Provide improved skid resistance and reduced noise

Tack coat: A tack coat is an application of a thin layer of liquid asphalt cement or emulsion or cutback asphalt on compacted granular base/subbase layer of pavement or existing pavement. Its purpose is to provide bonding between pavement layers and in between HMA lifts as well as bonding between existing pavement and HMA overlay.

Fog seal: A fog seal is a light application of a diluted asphalt emulsion. This is done to restore flexibility to an aged (oxidized) asphalt concrete pavement surface.

Slurry seal: A homogenous mixture of well graded fine aggregate, mineral filler, water and emulsified asphalt is used as slurry seal. Fine aggregate typically consists of natural or manufactured sand, slag, crushed rock fines, or other mineral material that conforms to ASTM-D1073. Mineral filler is typically the finer mineral aggregate material passing No. 200 sieve, and materials such as flue dust, fly ash, stone dust, hydrated lime, ammonium sulphate, or portland cement which conform to ASTM-D242 can be used. Depending on the grading of the aggregates used, the slurry seals are categorized into three types as given in Table 7.6 (ASTM-D3910).

Slurry seals are commonly used for rectifying pavement defects such as filling surface cracks and voids and improving skid resistance. The use of chemically active filler materials such as portland cement, ammonium sulphate, or hydrated lime improves the workability and controls the setting time of the slurry seal. Typical emulsified asphalts used in slurry seals are Grade SS-1h, CSS-1h, QS-1h and CQS-1h.

Type 1 consists of the finest aggregate gradation, and it is typically used for filling small surface cracks and voids or to provide a thin surfacing to improve skid resistance. It is also sometimes used for preparatory treatments of existing pavements prior to the placement of HMA overlays. Type 2 is the common type that consists of fine aggregates, coarser than Type 1, typically used to provide a thin surfacing to improve the skid resistance of pavements and to treat old pavements that exhibit raveling. Type 3 consists of the coarsest aggregate gradation among the three types and it is typically used for providing a wearing surface for pavements. Common uses, residual asphalt content of the asphalt emulsion, and application rates of the three different types of slurry seals given in ASTM-D3910 are summarized in Table 7.7.

Mix design: Trial mixes of the slurry are prepared using various weight proportions of dried aggregate, asphalt emulsion, filler, and water. Mixes are tested for consistency, set time, cure time, and abrasion resistance, adopting standard testing methods. Details regarding these tests are found in ASTM-D3910. Selection of a slurry seal mix for a pavement application is dependent on the traffic loading on the pavement and the climatic and curing conditions in the construction site.

Table 7.6 Grading of aggregates for different types of slurry seals from ASTM-D3910

Sieve size	% Passing by dry weight		
	Type 1	Type 2	Type 3
9.5 mm (3/8 in)	100	100	100
4.75 mm (No. 4)	100	90–100	70–90
2.36 mm (No. 8)	90–100	65–90	45–70
1.18 mm (No. 8)	65–90	45–70	28–50
600 μm (No. 30)	40-60	30–50	19–34
300 μm (No. 50)	25–42	18–30	12–25
150 μm (No. 100)	15–30	10–21	7–18
200 μm (No. 200)	10–20	5–15	5–15

(Based on ASTM-D3910)

Characteristics and uses of different slurry seal types

Table 7.7

	Slurry seal		
	Type 1	Type 2	Type 3
Residual asphalt content (weight % of dry aggregate)	10–16	7.5–13.5	6.5–12
Typical application rate, kg / m^2 (lb / yd^2)	3.3–5.4 (6–10)	5.4–8.2 (10–15)	≥8.2 (≥15)
Common use	Filling small surface cracks and voids or to provide a thin surfacing to improve erosion resistance/skid resistance	Providing a thin surfacing to improve skid resistance of pavement and to treat old pavement that exhibits raveling	Providing a new wearing surface for pavement

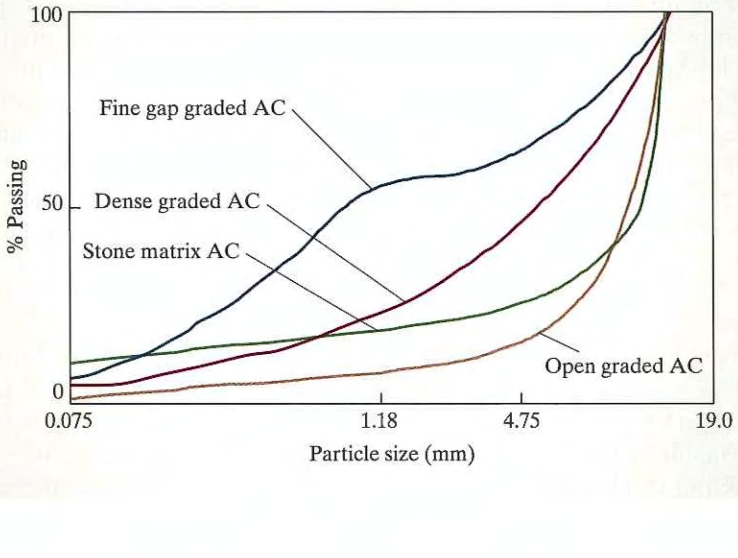
(Based on ASTM-D3910)

7.8 Asphalt Concrete

Asphalt concrete (AC) or HMA is the combination of asphalt cement and aggregates, with or without mineral filler, which are mixed together in a mixing plant. Asphalt cement is the binder, which generally ranges between 4 and 7 percent by dry mass in the AC mix. The AC is commonly manufactured, transported, spread, and compacted while hot to form a pavement layer and hence is referred to as hot mix asphalt (HMA) concrete. Depending on the grading of the aggregates that are used for making the asphalt concrete, it is categorized as dense graded, open graded, stone matrix, gap graded, and fine gap graded. Particle size distributions (PSD) of aggregates for some common types of asphalt concrete mixes are illustrated in Figure 7.24.

Illustration of aggregate PSDs for different types of asphalt concrete mixes (Figure is not to scale)

Figure 7.24



The reader may refer to Chapter 5 for details regarding the properties of aggregates and the grading requirement for Superpave HMA given in Table 5.6. Some general characteristics of these AC mixes in pavements are highlighted below.

Open graded asphalt concrete (OGAC): OGAC is characterized by a large proportion of coarse aggregate and only small amount of fine aggregate and filler. OGAC has relatively high air voids, generally in the 18–25 percent range, relying largely on mechanical interlock of aggregate particles for stability. They are permeable and hence less durable than dense type mixes. However, the durability is generally improved by an increase in asphalt film thickness around individual aggregate particles and the use of modified asphalt. When used as wearing course, the characteristics of OGAC include good skid resistance, good riding surface, reduced light reflection (glare), reduced tire noise, and reduced tire splash and spray.

Stone matrix asphalt concrete (SMAC): SMAC is also a gap graded AC mix but with a high proportion of coarse aggregate, providing an interlocking stone-on-stone skeleton that resists permanent deformation. The coarse aggregate skeleton is filled with mastic of asphalt filler and fine aggregate. When used as a wearing course, SMAC has the following advantages: high durability, low permeability, low traffic noise, high resistance to reflection cracking, and high rut resistance.

Dense graded asphalt concrete (DGAC): DGAC is a low air voids mix and is commonly used for constructing the bases of pavements. Mix design is usually engineered, and the methods used are discussed later in this section.

Fine gap graded asphalt concrete (FGGAC): FGGAC is a dense (low air voids) mix but with intermediate-sized fractions replaced by finer fractions. It may also contain more filler. FGGAC mixes rely on the stiffness of the fine-aggregate/filler/binder mixture for stability. When used in residential streets and other light-traffic applications, they provide a fine-textured surface and workable mix that is more readily compacted to low in situ air voids. The combination of low air voids and relatively high asphalt cement content provides an extremely durable surface as well as good resistance to fatigue cracking. However, they have poor resistance for rutting at high road surface temperatures. FGGAC mixes are generally not used in base applications due to higher cost and reduced stiffness.

7.8.1 HMA Design

Aggregate grading and the asphalt cement for the HMA for a pavement application are chosen based mainly on the availability, cost, traffic loading on the pavement, function of the particular HMA layer in the pavement structure, and temperature and other environmental conditions at the site. One of the purposes of mix design is to determine the *design asphalt cement content* for the selected aggregates and asphalt that could produce the HMA of desired characteristics. If the asphalt content is too low, the aggregate particles are not bound sufficiently by the asphalt, and therefore the HMA is less stable and not durable, leading to premature failure of the pavement. If the asphalt content is too high, the HMA is uneconomical, less stiff, and less stable. Hence, it could lead to larger permanent deformation under traffic loading and premature rutting failure of pavement.

HMA is designed traditionally in accordance to two methods: the Marshall (ASTM D1559) and Hveem (ASTM D1560) methods. Both these methods are empirical in nature and are based on preparing compacted samples of trial mixes, using the selected asphalt binder and aggregates and testing them under certain loading and environmental conditions. Extrapolating the mix design to a wide range of traffic loading and environmental conditions of pavements around the United States is difficult. Therefore, the Superpave HMA design method was introduced in 1993 after extensive research in the 1980s and early 1990s under the Strategic Highway Research Program (SHRP). However, both Marshall and Hveem mix design methods have been used successfully for several decades to design HMA of pavements which have performed well. The Marshall method is simpler, easy to test, and therefore more popular. The Marshall and Superpave methods are discussed in detail in this section. Volumetric properties of HMA are directly related to its performance, and all three mix design methods require volumetric analysis. The density and voids analysis of compacted HMA is discussed next.

7.8.2 Density and Voids Analysis of HMA

The asphalt cement coats the surface of aggregate particles during the mixing process of HMA. As discussed in Chapter 5, the aggregate particle used in the HMA generally has voids on the surface which are permeable for water. When dry aggregate is mixed with asphalt cement during the HMA manufacturing process, only parts of surface voids in the aggregate can get filled by asphalt as illustrated in Figure 7.25. The cumulative of the parts of voids that get filled with asphalt binder in HMA is referred as *absorbed asphalt.*

Another important term used in density and voids analysis of HMA is the *effective volume*: the summation of the volume of the solid aggregate and the volume of surface voids not filled with asphalt. Also, total asphalt minus the absorbed asphalt is the *effective asphalt*, either in terms of mass or volume.

In the mix design process, the volume of asphalt cement and the aggregates required are determined for manufacturing the HMA with the desired properties. But, volumes are difficult to be measured; therefore, the weights are often used, which are then converted to volumes with the use of appropriate specific gravities. Now examine the common specific gravity and other terms used in HMA design.

Effective specific gravity, G_{se}

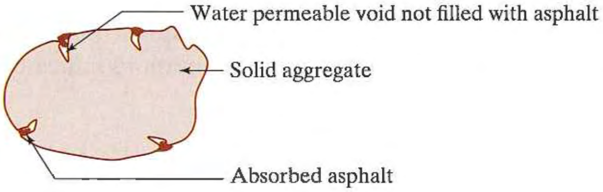
G_{se} is defined as

$$G_{se} = \frac{Dry\ mass}{Effective\ volume} \tag{7.7}$$

Where effective volume = volume of the solid aggregate + volume of surface voids not filled with asphalt cement (see Figure 7.25).

Illustration of absorbed asphalt and void not filled with asphalt

Figure 7.25



Theoretical maximum specific gravity, G_{mm}

A sample of loose HMA is prepared, adopting standard procedures as those used for other tests such as the Marshall Stability and Flow test. HMA is then cooled to the test temperature of 25 °C (77 °F), spread on a table, and the asphalt coated aggregate particles are separated by hand. The dry weight of the loose HMA specimen is measured. The specimen is then transferred to a vacuum bowl and covered completely with water. The vacuum bowl with the specimen and water is placed on a mechanical agitator, and vacuum is applied to the vacuum bowl thereby removing the air trapped in the specimen. Once all the air in the specimen is removed, the vacuum pressure is released and the bowl with the contents is suspended in water (without the lid used for connecting it to the vacuum pump). The submerged weight of the vacuum bowl with the specimen is measured. The submerged weight of the empty vacuum bowl (without the lid), commonly referred as the calibration weight of the vacuum bowl, also should be known for determining the submerged weight of the HMA specimen. The theoretical maximum specific gravity of the HMA (G_{mm}) is determined as:

$$G_{mm} = \frac{A}{A - (C - B)} \tag{7.8}$$

where

A = weight of the dry loose HMA specimen (g)
B = weight of empty vacuum bowl submerged in water (g)
C = weight of vacuum bowl with the specimen submerged in water (g)

Further details regarding the determination of G_{mm} discussed above can be found in ASTM D2041.

Effective specific gravity (G_{se}) that was discussed earlier is related to G_{mm} by the following equation:

$$G_{se} = \frac{100 - P_b}{\dfrac{100}{G_{mm}} - \dfrac{P_b}{G_b}} = \frac{P_s}{\dfrac{100}{G_{mm}} - \dfrac{P_b}{G_b}} \tag{7.9}$$

where

P_s = percent weight of aggregate
P_b = percent weight of asphalt cement
G_{se} = effective specific gravity of aggregate coated with asphalt cement
G_b = specific gravity of asphalt cement (typically 0.9–1.05)

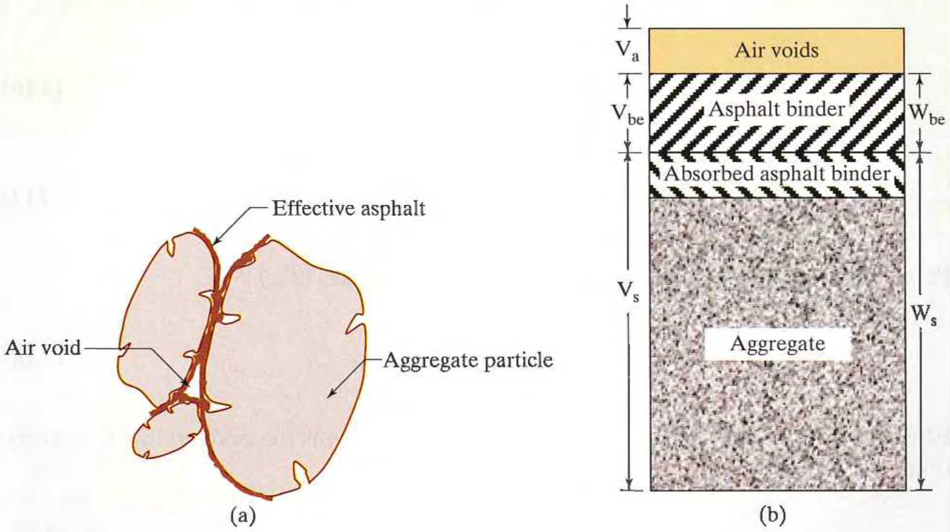
Compacted samples of trial mix of HMA are prepared and tested for strength and other properties during the mix design process. Moreover, coring from the HMA layer of an existing pavement could also be tested in the same fashion. A compacted sample of HMA typically consists of aggregates, asphalt binder, and air voids as shown in Figures 7.26a and 7.26b. Referring to the notations indicated in Figure 7.26b, volumes on the left side and weights on the right,

$$\text{Total volume of the sample of HMA, } V_m = V_a + V_{be} + V_s \tag{7.10}$$

$$\text{Total weight of the sample of HMA, } W_m = W_{be} + W_s \tag{7.11}$$

Figure 7.26

(a) Illustration of effective asphalt and air void in compacted HMA
(b) Volume and weight of the components of compacted HMA



The commonly used terms and their definitions are:

$$\text{Voids in total mix, } VTM = \frac{V_a}{V_m} \times 100 \tag{7.12}$$

$$\text{Voids in mineral aggregate, } VMA = \frac{V_a + V_{be}}{V_m} \times 100 \tag{7.13}$$

Voids filled with asphalt, VFA, is the percent of VMA that is filled with asphalt cement. Thus,

$$VFA = \frac{V_{be}}{V_{be} + V_a} \times 100 \tag{7.14}$$

where

V_a = volume of air voids
V_{be} = volume of effective asphalt binder
V_m = total volume of HMA
V_s = combined volume of aggregate and absorbed asphalt
W_s = combined weight of aggregate and absorbed asphalt
W_{be} = weight of effective asphalt

It is worth noting that the density of the aggregate, asphalt binder, and so forth can be calculated by multiplying the specific gravity with the density of water, which is typically 1000 kg/m^3 (1 g/cm^3, 62.4 lb/ft^3).

In terms of weight percentage and specific gravity:

$$VMA = 100 - \frac{G_{mb}P_s}{G_{sb}} \tag{7.15}$$

$$VTM = 100\left(1 - \frac{G_{mb}}{G_{mm}}\right) \tag{7.16}$$

$$VFA = 100\left(\frac{VMA - VTM}{VMA}\right) \tag{7.17}$$

Percent absorbed asphalt binder by weight of aggregate (P_{ba}) is

$$P_{ba} = 100 \times \left(\frac{G_{se} - G_{sb}}{G_{se} \times G_{sb}}\right) \times G_b \tag{7.18}$$

Effective asphalt cement is the total asphalt cement minus the percent lost to absorption; that is,

$$P_{be} = P_b - \frac{P_{ba}}{100} \times P_s \tag{7.19}$$

where

P_b = percent of asphalt binder by weight
P_s = percent of aggregate by weight
P_{ba} = percent of absorbed asphalt binder by weight of aggregate
P_{be} = percent of effective asphalt binder by weight of aggregate
G_{se} = effective specific gravity of aggregate coated with asphalt cement
G_b = specific gravity of asphalt binder
G_{sb} = bulk specific gravity of aggregate
G_{mb} = bulk specific gravity of the compacted HMA

The bulk specific gravity (G_{sb}) of aggregate is the ratio of the weight of a particular volume of aggregate to the weight of water of equal volume. Different procedures are used for coarse and fine aggregates for determining the bulk specific gravity, and the details could be found, for example, in ASTM C127 and ASTM C128.

When aggregate stockpiles of different sizes A, B, C, . . . having bulk specific gravity each of G_A, G_B, G_C, . . . are combined in weight proportion P_A, P_B, P_C, . . . respectively for a blended mix, then the average bulk specific gravity of the combined aggregate is given by,

$$G_{sb} = \frac{P_A + P_B + P_C + \ldots.}{\left[\dfrac{P_A}{G_A} + \dfrac{P_B}{G_B} + \dfrac{P_C}{G_C} + \ldots.\right]} \tag{7.20}$$

EXAMPLE 7.1

Aggregates from three stockpiles A, B, and C with bulk specific gravity each of 2.713, 2.698, and 2.721 are to be combined in weight proportion 25 percent, 50 percent, and 25 percent, respectively, for obtaining a blended aggregate. Calculate the average bulk specific gravity of the combined aggregate.

Solution:

$P_A = 25\%; P_B = 50\%;$ and $P_C = 25\%$ $G_A = 2.713; G_B = 2.698;$ and $G_C = 2.721$

The bulk specific gravity of the combined aggregate can be calculated using Eq. 7.20,

$$G_{sb} = \frac{25 + 50 + 25}{\left\{\frac{25}{2.713} + \frac{50}{2.698} + \frac{25}{2.721}\right\}} = 2.707$$

EXAMPLE 7.2

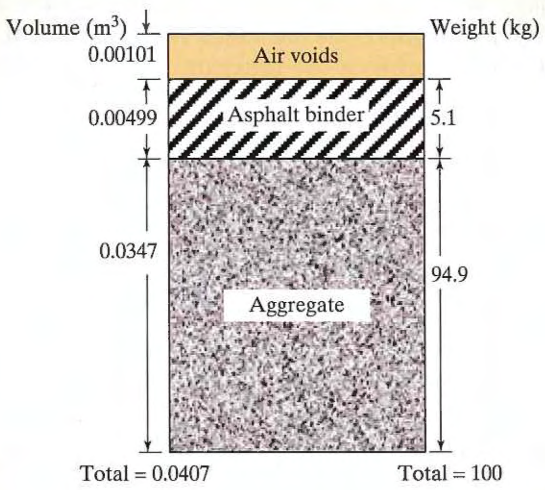
A compacted sample of HMA contains 5.1 percent asphalt by weight of total mix, and the bulk density of the HMA specimen is 2455 kg/m^3. The specific gravity of aggregate and the asphalt binder are 2.735 and 1.022, respectively. Determine the VMA, VTM, and VFA, neglecting absorption.

Solution:

Let us consider a sample of compacted HMA weighing 100 kg and determine its constituents in terms of weight as well as volume as shown in Figure 7.27.

Volume and weight of the components in the sample

Figure 7.27



Weight of asphalt cement (W_b) = 5.1 kg
Therefore, the weight of the aggregate = 100 − 5.1 = 94.9 kg
Since the bulk density of HMA is 2455 kg/m^3, the total volume of the sample (V_m) = 100/2455 = 0.0407 m^3

The volumes could be calculated using the specific gravities; thus

$$\text{Volume of the asphalt cement } (V_b) = \frac{5.1}{1.022 \times 1000} = 0.004990\ m^3$$

$$\text{Volume of the aggregate } (V_s) = \frac{94.9}{2.735 \times 1000} = 0.03470\ m^3$$

$$\text{Therefore, volume of air voids } (V_a) = 0.0407 - 0.03470 - 0.00499\ m^3$$
$$= 0.00101\ m^3$$

$$VMA = \frac{V_a + V_{be}}{V_m} \times 100 = \frac{V_a + V_b}{V_m} \times 100 \quad (V_{be} = V_b \textit{ since absorption is neglected})$$

$$\therefore\ VMA = \frac{0.00101 + 0.00499}{0.0407} \times 100 = 14.74\%$$

$$VTM = \frac{V_a}{V_m} \times 100 = \frac{0.00101}{0.0407} \times 100 = 2.48\%$$

$$VFA = \frac{V_{be}}{V_{be} + V_a} \times 100 = \frac{V_b}{V_b + V_a} \times 100 = \frac{0.00499}{0.00499 + 0.00101} \times 100 = 83.14\%$$

EXAMPLE 7.3

A compacted sample of HMA contains 5.5 percent asphalt binder by weight of total mix, and its bulk specific gravity is 2.431. The theoretical maximum specific gravity of the mix is 2.589, and the bulk specific gravity of the aggregate is 2.705. Determine the VMA, VTM, and VFA.

Solution:

Weight percentage of aggregate in mix, $P_s = 100 - P_b = 100 - 5.5 = 94.5$

$$\text{Hence, } VMA = 100 - \frac{G_{mb}P_s}{G_{sb}} = 100 - \frac{2.431 \times 94.5}{2.705} = 15.1\%$$

$$VTM = 100\left(1 - \frac{G_{mb}}{G_{mm}}\right) = 100\left(1 - \frac{2.431}{2.589}\right) = 6.1\%$$

$$VFA = 100\left(\frac{VMA - VTM}{VMA}\right) = 100\left(\frac{15.1 - 6.1}{15.1}\right) = 59.6\%$$

7.8.3 Marshall Method

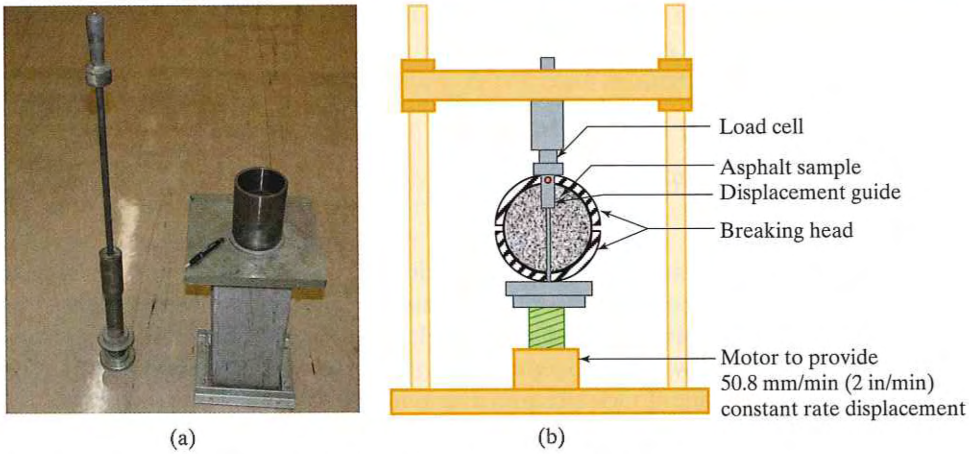
The Marshall mix design method initially involves the selection of aggregate, the asphalt binder, and the range of asphalt binder content for testing in the HMA trials. These selections are usually based on previous experience and/or guidelines given by American Association of State Highway and Transportation Officials (AASHTO), state departments of transportation, or similar organizations. It is worth noting that characteristics such as gradation, particle shape, and surface texture; cleanness and deleterious substances; durability and soundness; and specific gravity and absorption of available aggregates, discussed in Chapter 5, are also considered in the aggregate selection process. The next step is to prepare samples of trial mixes for the range of asphalt cement contents while the same aggregate blend is used for all HMA samples.

Sample preparation: Typically 101.6 mm (4 in.) diameter, 63.5 mm (2.5 in.) high cylindrical specimens of HMA with the asphalt cement content varying in steps of 0.5 percent by dry weight of aggregate are prepared adopting standard procedure (e.g., ASTM D1559). At least three specimens per asphalt cement content are tested. For example, three samples each at 4.0, 4.5, 5.0, 5.5 and 6.0 percent binder content are prepared for testing the binder range between 4.0 and 6.0, amounting to a total of fifteen samples. The highlights of the HMA sample preparation are:

- Aggregate and asphalt cement are heated to the specified temperatures separately and then mixed thoroughly.
- Mixing temperature is specified depending on the kinetic viscosity of the asphalt cement for the HMA.
- Sample is compacted in a standard-sized cylindrical mold.
- Compaction effort is either 35, 50 or 75 blows, depending on the design traffic category, using a standard hammer, falling 450 mm in distance (Figure 7.28a).

Stability and flow determination: The test specimen is extruded from the mold and cooled to 60 °C (140 °F), typically by immersing in a temperature-controlled water bath for a specified time. This is followed by stability-flow testing of the sample

(a) Sample preparing devices (b) Schematic of Marshall testing apparatus **Figure 7.28**



(a) (b)

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

in a loading machine, using standard-sized *breaking head* components placed on top and bottom curved faces of the sample as shown in Figure 7.28b. The sample is loaded at a constant displacement rate of 50.8 mm/min (2 in/min), and the load and the vertical diametric displacement of the specimen are monitored during the loading. The stability and flow are determined as follows: the maximum load resistance in N is referred to as *stability*; the total displacement of the specimen, in units of 0.25 mm (0.01 in) as the load is increased from zero to the maximum, is referred as *flow*.

Usually the asphalt cement content is varied in 0.5 percent increments from the minimum, ensuring that at least two binder contents are below the optimum and two above the optimum. Although the load cell and displacement transducer are shown in Figure 7.28b, the original Marshall method involved the use of a proving ring and dial-gauge for load and displacement measurements, respectively. Moreover, the standard method (ASTM D1559) discussed above is meant for aggregate up to 25 mm (1 in) nominal maximum size. Larger-sized cylindrical samples, 152.4 mm (6 in.) diameter and 108 mm (4.25 in.) high, prepared using a bigger hammer and adopting mechanized compaction, are to be used for HMA having larger size aggregate up to 37.5 mm (1.5 in.) nominal maximum size (ASTM D5581).

Usually the bulk density of the compacted HMA sample is also determined by measuring the weight of the prepared cylindrical sample in the air and when it is submerged in water.

$$\text{Bulk density of compacted HMA (g/cm}^3) = \frac{W_a}{W_a - W_w} \times \rho_w \qquad \textbf{(7.21)}$$

where

W_a = weight of sample in air (g)
W_w = weight of sample immersed in water (g), and
ρ_w = density of water (g/cm^3).

Bulk specific gravity of the compacted HMA (G_{mb}) is the ratio between the above bulk density and the density of distilled water at the same temperature. Since the density of water is approximately 1.0 g/cm^3, G_{mb} is numerically equal to the above bulk density.

Selection of the best HMA for a pavement construction project is made considering the following:

- Stability and air voids
- Durability
- Flexibility
- Economy
- Construction considerations
- Size of project

Marshall method test data exhibits a poor relationship with actual field performance data, primarily because of the differences in the compaction procedure adopted in the lab compared to that performed in the field, as well as temperature and environmental conditions. A new approach is to use a gyratory compactor to prepare samples and determine the resilient modulus and dynamic creep properties.

7.8.4 Superpave Mix Design

The ***Su***perior ***per***forming asphalt ***pave***ment (Superpave) mix design approach was the main outcome of the SHRP, and it was intended to replace the Marshall and Hveem methods. Asphalt binder and aggregate selection are incorporated in the Superpave mix design approach. Superpave is an extensive mix design approach in which the climate condition at the site, pavement traffic loading, traffic speed, and the layer in which the particular HMA is used within the pavement structure are also considered. The approach is quite comprehensive, and a detailed discussion can be found in Asphalt Institute's Superpave Manual (SP-2).

Key components of the Superpave mix design approach are:

- Performance-based asphalt cement specifications that correspond to the climate and pavement temperatures in the field
- Volumetric mix design and analysis
- An improved laboratory gyratory compaction method for trial HMA samples that reflects field conditions.

The steps involved in the Superpave mix design method are (Harman et al., 2002):

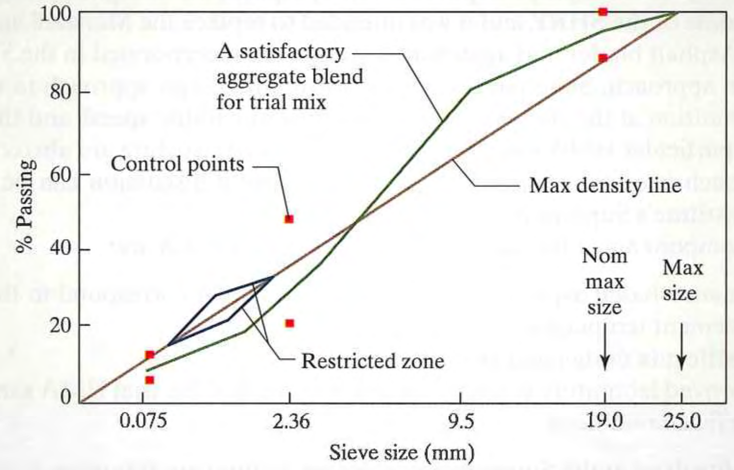
- Aggregate selection
- Asphalt cement selection
- Sample preparation of trial
- Density-voids analysis and selection of design aggregate structure and optimum asphalt binder content
- Moisture susceptibility evaluation

Aggregate selection: Aggregate selection should satisfy *source* and *consensus* requirements according to the Superpave mix design. The source requirements include Los Angeles (LA) abrasion, cleanness and deleterious substances, and durability and soundness by source aggregate stockpiles. Consensus requirements include gradation, angularity of both coarse and fine aggregates, clay content, and flat and elongated particles. Details regarding these properties and the corresponding tests for determining them are given in Chapter 5. The Superpave mix design has specified limits on these properties depending on the *design traffic* and layer thickness in the pavement.

The Superpave mix design has specified *gradation control points* and a restricted gradation zone to avoid excessive rounded natural sand as highlighted in Figure 7.29. Aggregates from different stockpiles are blended, using methods discussed in Chapter 5 to obtain a satisfactory gradation (see also Table 5.6). Material passing 0.075 mm (#200) sieve is referred as *dust* in Superpave mix design, and restrictions are placed on the *dust to asphalt binder* (i.e., $\frac{P_{0.075}}{P_{be}}$) *ratio*, where $P_{0.075}$ = proportion by weight of aggregate material passing 0.075 mm (#200) sieve, and P_{be} = weight proportion of effective asphalt binder content. The dust-to-binder ratio normally ranges from 0.6 to 1.2.

Asphalt binder selection: Superpave uses its own performance grading (PG) system and related specifications discussed earlier in Section 7.2.4 for asphalt selection. A PG asphalt binder is selected on the basis of design maximum and minimum pavement temperatures, and algorithms have been developed by SHRP researchers to convert high and low air temperatures to maximum and minimum pavement temperatures. Seven-day average high air temperature and one-day low temperature

Figure 7.29 Illustration of a Superpave aggregate gradation chart



are commonly used for determining the maximum and minimum pavement temperatures, which are then subject to reliability concepts for selecting the suitable PG asphalt binder.

Sample preparation of trial blends and performance testing: This involves preparing trial mixes of aggregate and the asphalt that meet the Superpave requirements. The asphalt content range is chosen on the basis of estimated optimum binder content such that points above and below the final optimum binder content are investigated. Specimens are compacted using a Superpave gyratory compactor (SGC) at the specified temperature. The SGC used in sample preparation simulates the field roller compaction of HMA. Specimens are prepared in a SGC with a gyration angle of 1.15° under 600 kPa (87 psi) vertical pressure. The specimens typically measure 150 mm (6 in.) diameter and 115 mm (4.5 in.) high. Superpave compaction criteria are based on the number of gyrations corresponding to three critical stages during compaction; namely $N_{initial}$, N_{design}, and $N_{\max}$, and they are based on expected pavement design traffic. $N_{initial}$ is a measure of mixture compactability during construction, N_{design} corresponds to compaction of the mix at completion of construction, and $N_{\max}$ corresponds to ultimate density of the mix after the pavement has served the traffic for a number of years.

Density-voids analysis and selection of design aggregate structure and asphalt binder content: Density and volumetric properties of the mixture are analyzed, using the same procedures as discussed earlier in Section 7.7.2. The best aggregate blend is selected as the design aggregates structure, based on desired performance criteria for the mixture. The binder content that results in 4 percent air voids at N_{design} is selected as the optimum binder content.

Moisture Susceptibility Evaluation: Moisture susceptibility of HMA is tested, typically using AASHTO T 283 (Resistance of compacted bituminous mixture to moisture induced damage). This test is performed on specimens prepared at design binder content and 7 percent air voids and conditioned by vacuum saturation followed by freezing and thawing. Conditioned and unconditioned specimens are tested in a modified Marshall stability apparatus for tensile strength, and the ratio between them is determined. Superpave has imposed a minimum of 0.8 for this tensile strength ratio.

Note: The Superpave mix design method is relatively new and is proven to be economically beneficial. Hence, it is widely used across the United States by the state departments of transportation. Research is ongoing to further improve material behaviors and extending service life. Many highway agencies are currently performing modified versions of the traditional tests (e.g., the Marshall stability and flow tests) also on samples prepared using Superpave gyratory compactor for further insight into material behaviors.

7.8.5 HMA Production

The HMA is produced by mixing aggregates, filler, and asphalt cement in a mixing plant. In most cases, the aggregate and filler are heated and mixed with a hot asphalt to produce hot mix asphalt (HMA) concrete. The HMA concrete is then transported to the road construction site, spread, and compacted while hot. However, in certain situations, the HMA may be placed cold where unheated aggregate is mixed with a cold or warm cutback asphalt binder or with an asphalt emulsion, commonly referred as *cold mixing.* Alternatively, the aggregate may be dried, heated, and mixed with a warm cutback asphalt binder, referred as *warm mixing.*

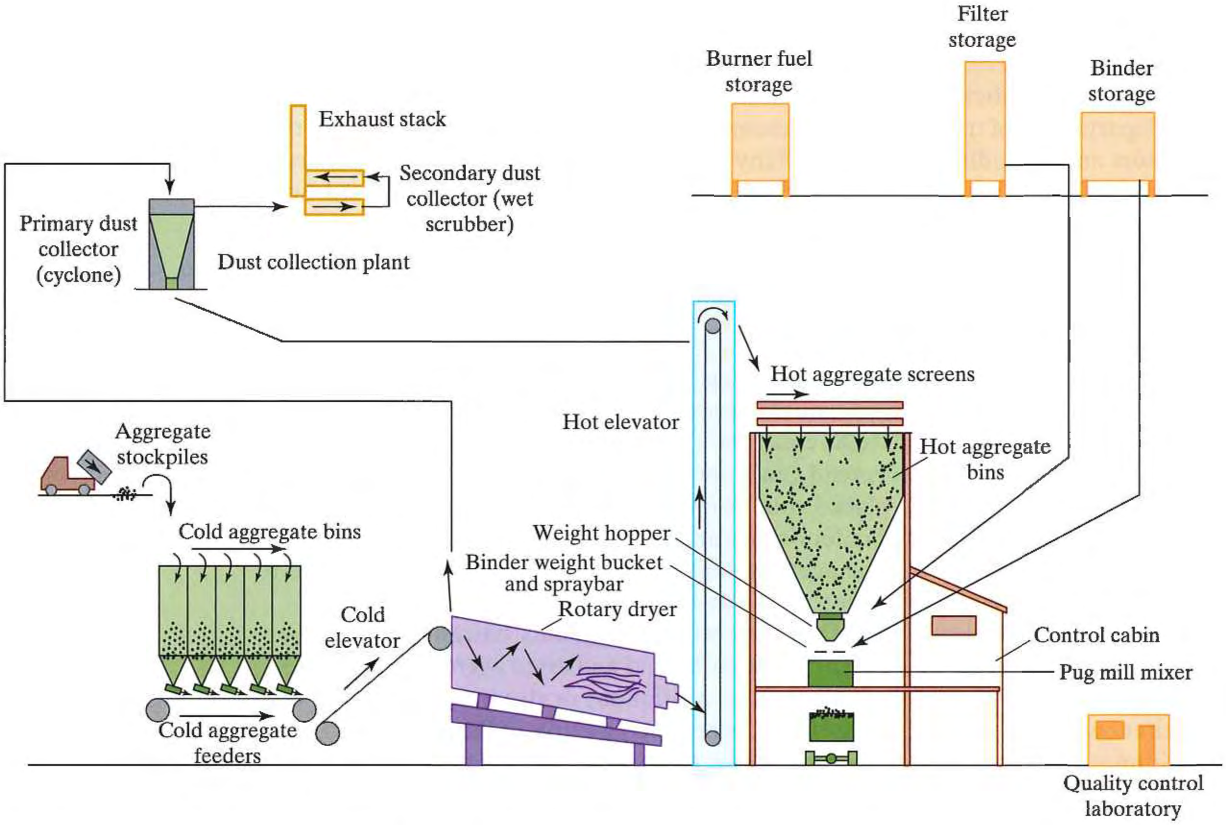
There are two basic types of HMA concrete mixing plants: batch and continuous or drum mixing plants. In batch mixing plants, heated and dried aggregates are mixed with the binder in a pug mill as a batch. In continuous or drum-mixing plants, the constituents pass through a pug mill or drum mixer continuously. Drum-mixing plants are simpler than conventional batch mixing and generally have the advantages of lower initial cost, reduced maintenance costs, less atmospheric pollution, and greater portability (Underwood, 1995). However, batch mixing plants are the most common type around the world, which offers the advantages of highest level of flexibility to change HMA as required to achieve superior quality. The period for the whole batch mixing typically ranges from 40 to 50 seconds, and a 3 tonne mixer could produce about 320 tonnes of HMA per hour.

Batch Mixing Plants

Aggregates are first transported from stockpiles and loaded into the cold feed bins of different aggregate sizes in a batch plant (Figure 7.30). The number of bins needed will depend on the mix to be produced and the aggregate sizes that are available. For example, only one bin may be sufficient for a plant to produce patching cold mix, whereas six or more bins may be needed for a plant to produce a dense-graded HMA.

The different-sized aggregates from the bins are then proportioned out by the cold aggregate feeders so that the required aggregate grading is attained. The proportioned aggregate combination is then fed into the rotary drier where it is heated to the required mix temperature while the moisture and dust are extracted. The dry hot aggregate is then elevated to pass through hot sieves which separate the aggregates and charge into hot aggregate storage bins. The hot aggregates, filler, and binder are then weight-batched automatically into a pug mill mixer, where they are mixed thoroughly and then discharged into a surge silo or directly into a truck for delivery to the road construction site.

Figure 7.30 Flow diagram of a typical HMA batch mixing plant



(Based on Underwood, R.T. (1995). Road Engineering Practice. Macmillan Education Australia Pty Ltd.)

Drum Mixing Plants

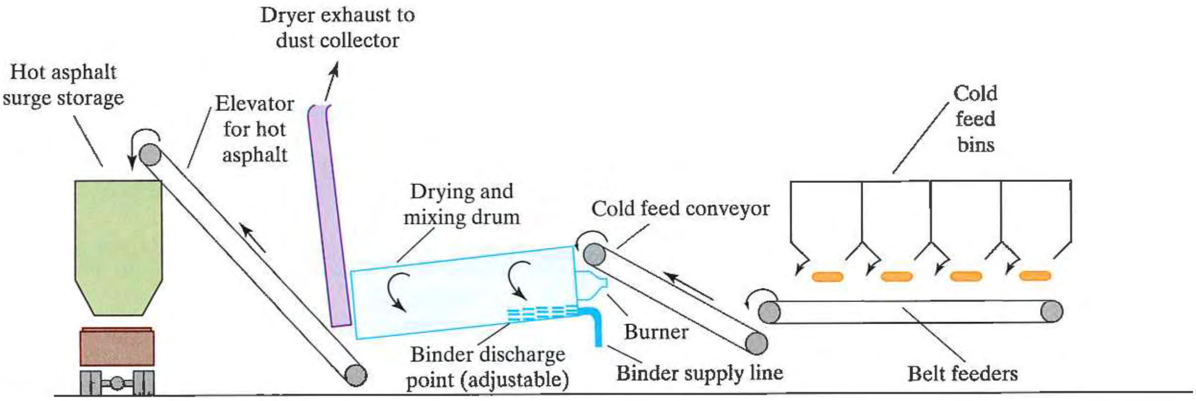
In a drum mixing plant (Figure 7.31), the different-sized aggregates from the cold feed bins are proportioned out to attain the required aggregate grading. The proportioned aggregates are then continuously fed through a calibrated gate at one end of the pug mill or drum mixer while the asphalt binder is introduced continuously at a fixed rate. Mixing takes place as the material moves through the pug mill or drum mixer, and the mix is then discharged at the other end.

Asphalt Storage

Sometimes there is a need to store fresh HMA, and surge silos are generally used for this purpose. Surge silos are usually insulated circular or rectangular structures, with the lower part being conical or wedge-shaped. The HMA is fed in at the top of the silo, and its internal arrangement is designed such that segregation is minimized. Their capacity is normally up to 400 tonnes, and the length of storage time is usually less than 24 hours. The advantages of using surge silos include (Underwood, 1995) that the plant can produce HMA at a continuous fixed rate, thereby operating more

Schematic of a typical HMA Drum mixing plant

Figure 7.31



(Based on Underwood, R.T. (1995). Road Engineering Practice. Macmillan Education Australia Pty Ltd.)

economically, fluctuations in HMA demand can be smoothed out, and loading HMA in trucks is rapid. Hence the cycle times of trucks are reduced, and HMA can be stored for delivery outside normal plant operating times.

7.8.6 Characterization of HMA for Pavement Design

The HMA is laid and compacted in small lifts for constructing the base and surface courses of pavements as discussed earlier (e.g., Figures 7.7 to 7.11). The pavement layers are subjected to repetitive loading due to traffic movement. Application of large numbers of repetitive loads due to vehicle axle movement could lead to fatigue cracking and excessive accumulated permanent deformation or rutting failure. These are the primarily modes of failure in pavements, and models have been developed for determining the fatigue life and permanent deformation accumulation characteristics. These models are based on the dynamic modulus and accumulation of permanent deformation characteristics under repetitive loading conditions.

The modulus of a particular AC mix is dependent on the temperature and the traffic speed. Many factors that influence the modulus of AC mix are summarized in Table 7.8, and most of these factors are considered during the mix design process. In reality, HMA concrete shows viscoelastic behavior, but in most pavement design methods, it is approximated to be an elastic solid material for the range of loading and environmental conditions.

Two common tests that are used for characterizing AC in pavement design are the indirect diametrical tensile (IDT) test and the triaxial compression test under repetitive loading. Wheel-tracking tests on AC slab samples (Figure 7.32) or simulated pavement structure for studying the permanent deformation accumulation behavior and triaxial compression testing under sinusoidal loading for determining the dynamic modulus are also carried out by many highway agencies.

The repetitive load triaxial compression test equipment used for determining the resilient modulus and permanent deformation accumulation characteristics is the same as that used for unbound granular materials discussed in Chapter 5 (see

Table 7.8 Factors affecting the stiffness of HMA and the effect of increasing the factor values

Factor	Change in AC stiffness due to increase in factor
Proportion of coarse angular particles	Increase
Density	Increase
Asphalt viscosity	Increase
Stress level	No change
Age	Increase
Extent of cracking	Decrease
Efficiency of mixing	Increase
Asphalt binder content	Increase then decrease
Air voids (%)	Decrease
Temperature	Decrease
Rate of loading	Increase

(Based on Austroads 2012)

Figure 5.12) and thus not discussed further here. It should be noted, however, that the repetitive compressive loading and confinement stress levels are different for AC compared to that used for unbound granular materials.

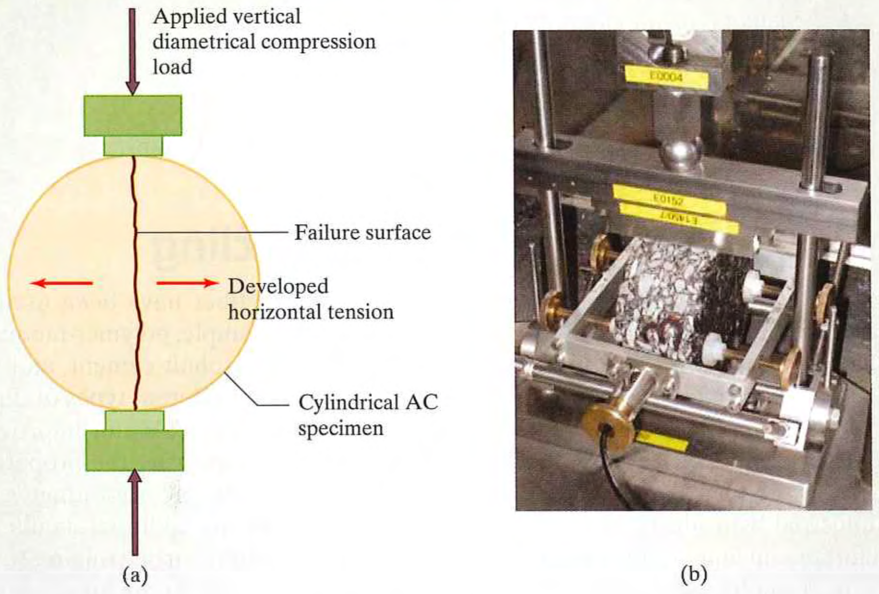
A schematic of the IDT testing arrangement and an AC sample undergoing the test are shown in Figure 7.33. The sample for IDT testing is usually prepared, using a gyratory compactor. Alternatively, core samples of AC from existing pavements could

Figure 7.32 Typical wheel-tracking test equipment

(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

IDT testing: (a) schematic of the test (b) an AC sample being tested

Figure 7.33



(Courtesy of Rajah Gnanendran, ADFA School of Engineering and Information Technology)

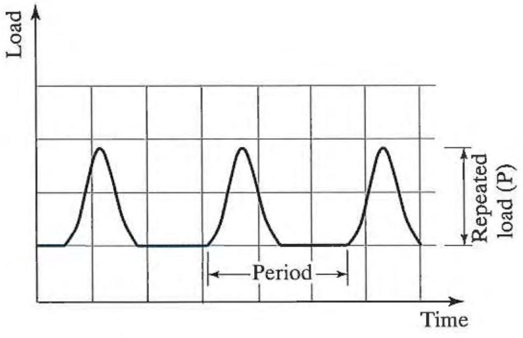
also be tested. The resilient modulus of AC is usually determined from repetitive load IDT testing (ASTM D7369, AS/NZS 2891.13.1). A repeated vertical compressive force is applied along the vertical diametrical plane, and the horizontal displacements at mid-height through the horizontal diameter are measured, using LVDTs on either side of the sample. Typical variation of the applied vertical compressive load with time is shown in Figure 7.34. The horizontal displacement is monitored with time.

The resilient modulus is calculated, using the equation below:

$$E = \frac{P(v + 0.27)}{t \times \delta_h} \tag{7.22}$$

Typical load–time variation of repeated load IDT testing

Figure 7.34



Where:

E = estimated resilient modulus, MPa (psi)
P = applied repeated load, N (lb)
v = Poisson's ratio, typically 0.35 at 25 °C (77 °F)
t = thickness of specimen, mm (in.)
δ_h = recovered horizontal tensile deformation in a load cycle, mm (in.)

7.9 Use of Additives and Recycling

Different types of additives such as rubber, plastic, and fiber have been used to improve or modify the properties of asphalt or AC. For example, polymer-modified asphalt has been produced by the addition of rubber to asphalt cement, and the current trend is to use rubber obtained from scrapped tires. Different types of fibers (e.g., polyester, polypropylene, steel, and fiberglass) are added to AC for improving its tensile properties. Extensive research is underway for improving the properties of asphalt products as well as to derive economic benefits by incorporating waste or industrial byproducts. Moreover, recycling the AC with the addition of suitable asphalt cement and/or other additives has also been successfully carried out in certain road and airfield pavement construction projects. The potential for further research and development on asphalt products is enormous, and more new and improved materials are expected in the future.

7.10 Summary

1. Asphalt is a viscous liquid at high temperature but solid at normal air temperature, and it is mostly produced from crude oil. It adheres strongly to most materials. Therefore, it is used as the binder for asphalt concrete (AC) mix, which is generally produced at high temperature and laid and compacted while hot for constructing pavements. However, liquid asphalt products such as emulsified asphalts are also produced, which are used in AC mix as well as for surface treatment and sealing work.
2. Asphalt cement products are generally classified in terms of *standard penetration* and *viscosity*, and they are graded in many ways. The most common grading methods used in pavement construction are standard penetration, viscosity, aged residue, and performance grading. Performance grading is the most recent development, and the Superpave AC mix design method is tied to this grading.
3. Cutback asphalt, emulsified asphalt, foamed asphalt, and tars are other common asphalt products used in pavements.
4. Emulsified asphalts are classified into four types as slow, medium, rapid, and quick setting. Within the type, they are graded according to the viscosity or the characteristic of the residual asphalt. Selection of the type and grade of asphalt emulsion is dependent on whether it is to be used for asphalt concrete mix or surface treatment.
5. Consistency, rate of curing, and durability of asphalt cements are important considerations for their use in pavements. Consistency is commonly assessed through viscosity or standard penetration, depending on whether the asphalt cement is in

liquid or solid/semi-solid state. Oxidation, volatilization, and age hardening are the main durability concerns of asphalt products used in pavements, which could lead to loss of plasticity, cracking, changes in surface roughness, and subsequent failure.

6. Asphalt cement is tested for various properties, and the common tests and the properties determined are ductility test to measure the extent to which it could be deformed before breaking; specific gravity test, flash point test, and direct tension test for determining the failure tensile stress and strain; bending beam rheometer test for determining the flexural creep stiffness; and the dynamic shear rheometer test for determining the complex shear modulus and phase angle.
7. Pavement construction and maintenance generally involve sprayed work and asphalt concrete work. Priming, sealing, primer sealing, chip sealing, tack coat, fog seal, and slurry seal are the common types of sealing work performed in pavement construction and/or maintenance. Materials and their application rates are selected or engineered to suit the need and the environmental conditions at the site.
8. Asphalt concrete (AC) is the combination of asphalt cement and aggregates, with or without mineral filler, mixed together in a mixing plant. Marshall, Hveem, and Superpave are the three common methods used for AC mix design, which aims to determine the optimum binder content. Each method uses different compaction method for compacting trial samples for testing. However, all three methods involve similar density and volume analysis.
9. Superpave mix design method is quite elaborate and uses performance grading for the asphalt binder, which in turn uses the design maximum and minimum pavement temperatures. The Superpave gyratory compactor is used for preparing trial samples. Also, the Superpave method involves the selection of *design aggregate structure* and the evaluation of moisture susceptibility of the designed mix.
10. Fatigue and excessive permanent deformation (rutting) are the two main modes of failure of AC in pavements. The resilient modulus of AC is needed for designing the pavement structure, and it is commonly determined from repeated load IDT testing.

Exercises

1. What are the desirable characteristics of asphalt cement?
2. Discuss the differences between asphalt and tar.
3. Describe three different methods of grading asphalt cements.
4. What is emulsified asphalt? Discuss the different types of emulsified asphalts and their uses.
5. Explain what is meant by temperature susceptibility of asphalt cement and the temperature effect on AC.
6. What is cutback asphalt? Between cutback asphalt and emulsified asphalt, what is preferred for pavement application and why?
7. Explain how the standard penetration test is performed on asphalt.

8. Describe the engineering aspects of the following tests:
    - a. Dynamic shear rheometer test
    - b. Bending beam rheometer test
    - c. Penetration test
9. Explain how short-term aging in asphalt cement is simulated in the lab.
10. Name three different types of sprayed sealing methods and explain the differences and advantages.
11. What is a slurry seal? Discuss its common use and briefly describe how a slurry seal mix is designed.
12. Define the following terms in relation to asphalt concrete:
    - a. Air voids
    - b. Voids filled with asphalt
    - c. Voids in mineral aggregate
    - d. Voids in total mix
    - e. Effective specific gravity
13. Describe the Superpave mix design process, highlighting the critical steps involved in the method.
14. A compacted sample of HMA contains 5.2% asphalt binder by weight of total mix, and the bulk density of the AC specimen is 2448 kg/m^3. Aggregates from three stockpiles with bulk specific gravity each of 2.715, 2.695, and 2.725 are to be combined in weight proportion 50%, 25%, and 25% respectively for the mix. The specific gravity of asphalt binder is 1.025. Neglecting the absorption, determine the VMA, VTM, and VFA.
15. A compacted sample of AC contains 5.3% asphalt binder by weight of total mix, and its bulk specific gravity is 2.428. The theoretical maximum specific gravity of the mix is 2.592, and the bulk specific gravity of the aggregate is 2.703. Determine the VMA, VTM, and VFA of the AC.
16. The following data was obtained from the testing of a core sample of AC: The weight of the sample in air is 1200 gm, and its volume is 680 cm^3. The bulk specific gravity of coarse aggregate, fine aggregate, and filler are 2.68, 2.74, and 3.10 respectively, and the specific gravity of asphalt binder is 1.02. If the weight proportions of asphalt, coarse aggregate, and filler are 6.4%, 60%, and 10% respectively, calculate the volume proportions of air, asphalt binder, and mineral aggregate in the AC (Note: Mineral aggregate = Coarse aggregate + Fine aggregate + Filler).

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CHAPTER 8

The Sydney Opera House is a reinforced concrete structure. Its shells consists of precast concrete ribs joined together by bolts and prestressing cables. Courtesy of R. Tuladhar, James Cook University, Australia

Cement and Concrete

8.1 Introduction

Concrete is the most used construction material in the world and the second most used material by humankind, second just to water. Every year, 25 billion tonnes of concrete are poured around the world. Owing to its strength, longevity, and ease in construction, concrete is widely used in construction of most of the civil engineering structures, including residential houses, high-rise buildings, industrial buildings, bridges, roads, and dams. The sheer volume of concrete used in the construction industry makes it one of the most important civil engineering materials.

Concrete is essentially a mixture of water, fine aggregates (sand), coarse aggregates (gravel or crushed stone), and cement. Apart from these basic constituent materials, other *admixtures* are often added to the concrete mix to alter its properties. Fresh concrete, obtained by mixing these materials, is workable and can be molded into any desired shapes. This unique trait of concrete gives it the required flexibility to construct wide range of structures. Within a few hours of mixing, cement undergoes a chemical reaction with water, forming a hardened paste. This reaction is generally known as the *hydration of cement*. The hardened paste binds aggregates, forming a hard, stone-like material called concrete. The hydration reaction may continue for weeks, months, and years under suitable conditions, increasing the strength of concrete with time. The properties of fresh and hardened concrete are governed by the type and amount of constituent materials used in the concrete mix, handling and placing of the mix, and curing of the concrete structure after being cast.

8.1.1 History of Concrete

The concept of using concrete-like material in construction dates back to the ancient Egyptians and Greeks. They used lime mortar with stone pieces in their constructions. However, the Romans were the first to develop *pozzolanic* concrete. They mixed lime and volcanic ash with water to form cementitious material, which they used as mortar for their stone constructions. Romans obtained volcanic ash from a village called Pozzuoli, near Naples in Italy; the term pozzolana or pozzolanic material refers to this place. They combined the pozzolana-based mortar with sand and stone pieces to produce strong and weather-resistant construction material, often referred to as Roman concrete. Many of the remarkable Roman structures like the Pantheon and the Colosseum were built using this material. The Pantheon (Figure 8.1) in Rome, built in 126 CE, is still the largest unreinforced solid concrete dome in the world. This underscores the strength and longevity of the concrete structures.

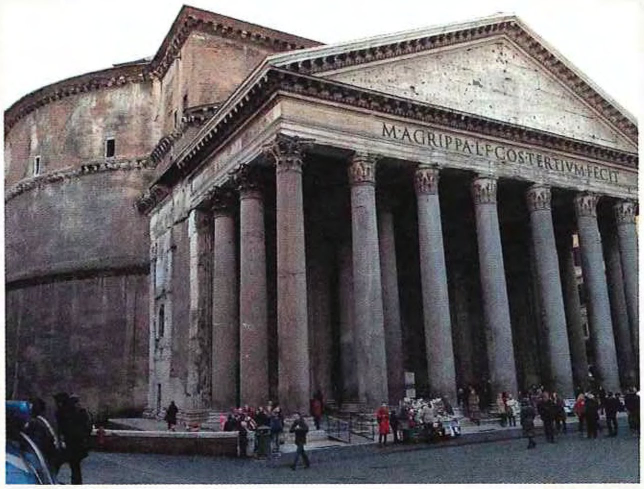
Modern-day concrete mostly uses *portland cement*, manufactured by heating a mixture of crushed limestone and clay to a sufficiently high temperature in a furnace. Portland cement is a hydraulic cement, which means it reacts with water to form water-insoluble hardened material. The manufacturing process and chemistry of portland cement are described in detail in Section 8.2.1. Portland cement is mixed with water and aggregates to produce concrete. In modern days, the type and proportion of constituent materials in concrete are designed and controlled carefully to achieve the desired strength and durability.

8.1.2 Advantages of Concrete

Concrete is widely used as construction material in the world because of its numerous advantages. The raw materials required for producing concrete are available abundantly in most of the world and are comparatively cheaper than other construction materials. Concrete is also a very versatile construction material. Fresh concrete is workable and moldable and can be cast into various forms and shapes, which gives flexibility in construction.

The Pantheon in Rome, built in 126 CE, is the largest unreinforced solid concrete dome in the world

Figure 8.1



(Courtesy of M. Adhikari)

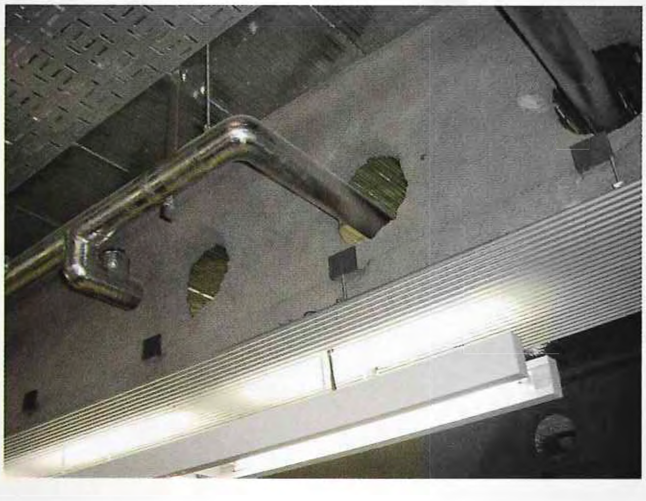
Another major advantage of concrete is its fire resistance. In comparison with other commonly used construction materials, concrete has the best fire resistance. It neither burns nor emits any toxic gases or molten particles when exposed to fire. Concrete is often used to construct fire shields and to protect other structural materials from fire damage. Figure 8.2 shows a steel beam encased in concrete to protect it from fire damage. Because of their inherent fire-resisting properties, concrete structures (unlike steel and timber structures) do not require any additional measures to prevent fire damage.

Concrete structures, if designed and constructed properly, are also resistant to water and weather. They sustain minimal damage in the event of flooding and can also be restored much more easily compared to other structures. They are durable, and if designed properly, can withstand tornados and significant earthquake forces as well. Roman concrete structures like the Pantheon and the Colosseum are again the testament of the longevity of concrete structures. Other distinct advantages of concrete are termite resistance, high thermal mass, and excellent acoustic performance. Concrete structures require minimal extra expenditure for maintenance compared to steel and timber structures, reducing the operational cost of the concrete structures.

8.1.3 Limitations of Concrete

Concrete has some inherent limitations, which should be taken into consideration while using it as a construction material. Concrete is very strong in compression; however, it is very weak in tension. Reinforcing steel is, therefore, embedded at strategic locations in concrete members to take the tensile stresses. Concrete combined with reinforcing, called reinforced concrete, is a very effective and versatile composite material. The reinforcing steel provides tensile strength and ductility (ability to withstand large deflections without failure) to concrete structures. On the other hand, concrete protects the reinforcement from fire and corrosion damage.

Figure 8.2 A steel beam encased in concrete to prevent fire damage



(Courtesy of R. Tuladhar, James Cook University, Australia)

Concrete construction requires expensive formworks for casting and molding concrete into required shapes. The cost of erecting formworks can be as high as 30 to 60 percent of the total cost of the construction. Use of *precast concrete* technologies, where concrete members are cast in factories and delivered to the site, can eliminate the need for formworks, saving cost and speeding up the construction.

Concrete members are usually bulkier than steel and timber ones and can significantly increase the dead load of the structure. *High-strength concrete* is often used in high-rise buildings and long-span bridges to reduce the size of the structural members.

Concrete structures can sustain time-dependent deformations such as *creep* and *shrinkage*. Creep is the increase in deformation in a structure at sustained loading. Shrinkage, on other hand, is the decrease in volume of the member due to the loss of absorbed water from the concrete surface. Creep and shrinkage in concrete structures can result in excessive cracking and deflection of the structural members under service load conditions. The control of creep and shrinkage in concrete is further discussed in Sections 8.6.5 and 8.6.6, respectively.

8.2 Constituents of Concrete

As discussed earlier, principal constituent materials of concrete are cement, aggregate (fine and coarse), and water. The properties of fresh and hardened concrete are significantly influenced by the type and proportion of the constituent materials. The constituent materials should be selected carefully to produce desirable concrete to meet strength and durability requirement of the structure. Additional admixtures are often added to the concrete mix to achieve specific desired characteristics.

8.2.1 Portland Cement

Cement, in general, is the binding material that reacts with water to form hardened paste that binds aggregates and other materials together. Joseph Aspdin, an English bricklayer and inventor, was the first person to patent portland cement in 1824. This cement was produced by heating the mixture of crushed limestone or chalk and clay in a furnace at a sufficiently high temperature to expel all the carbon dioxide. He coined the name portland cement after Portland limestone because the concrete produced using portland cement resembled Portland limestone—a popular building material during that time. In 1845, Issac C. Johnson invented the prototype of the modern portland cement by burning limestone and clay at a very high temperature until granular materials with different chemical composition, called *clinkers*, were formed. Clinkers were then ground to produce fine cement. The resulting portland cement is called hydraulic cement as it reacts with water to form a hardened cement paste which is insoluble in water. ASTM C150 (2011) defines portland cement as the hydraulic cement produced by pulverizing clinkers consisting of hydraulic calcium silicates and containing calcium sulfate.

Manufacturing Portland Cement

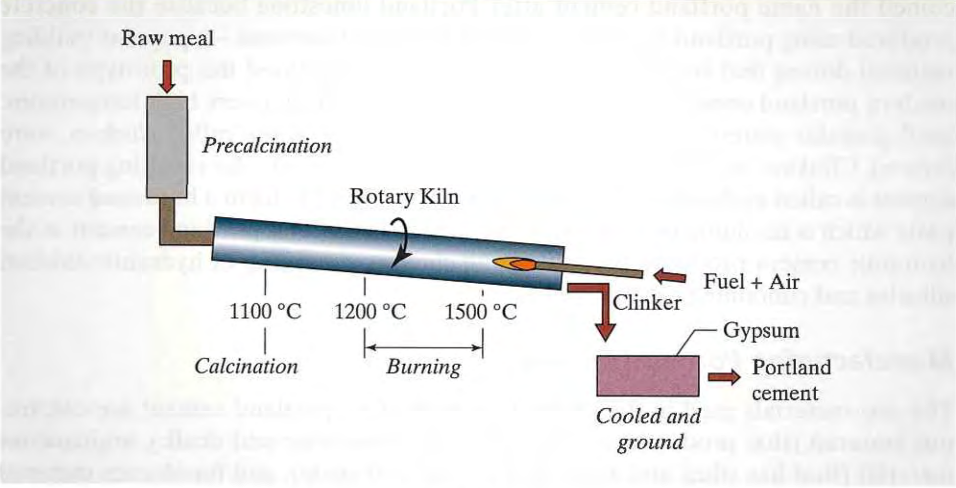
The raw materials used in the production of modern portland cement are calcareous material (that produces calcium oxide, like limestone and chalk), argillaceous material (that has silica and alumina, like clay and shale), and ferriferous material (iron ore). The first step in production of portland cement is preparation of these raw materials. Since limestone and clay are the main raw materials used in production of cement, most of the cement plants are located near the sources of these materials to reduce transportation time and cost.

Portland cement can be manufactured either by a *wet process* or a *dry process*. In a wet process, the raw materials are ground in presence of water in wash mills. The slurry, thus formed, is then fed into a rotary kiln. In the dry process (Figure 8.3), crushed raw materials are dried, ground in dry condition in ball mills to form fine powder, known as *raw meal*. The raw meal is preheated to about 800 °C and is often fed into precalcination vessels where 90 percent of the calcium carbonate compounds decompose into calcium oxide and carbon dioxide. The use of precalcination vessels improves the efficiency of the cement manufacturing process. The mixture is then fed into a heated rotary kiln.

It is easier to grind wet raw materials than dry ones. Moreover, in the wet process, it is easier to control the flow of the slurry in the kiln than feeding dry, fine raw materials. A major drawback of the wet process is that it is very energy intensive as the slurry needs to be dried before it is burned to form clinkers. The wet process also requires larger kilns as the water in the slurry needs to be evaporated. Most of the modern cement plants thus use the dry method, which is more energy efficient. As the raw mix is preheated in the dry process, shorter kilns can be used in the dry process compared to the wet process.

The dried ground raw meal is burned in a rotary kiln at a very high temperature (1400 °C to 1500 °C). The flow of raw meal is controlled by the slope and rotation of the kiln. At this high temperature, raw materials chemically react, melt, and recombine to form dark grey lumps 3 to 25 mm (0.1 to 1.0 in.) in diameter, known as cement clinkers (Figure 8.4). The cement clinkers are then cooled and ground in a ball mill to the required degree of fineness to produce cement. Gypsum ($CaSO_4.2H_2O$), 3 to 5 percent, is added during the grinding process. Gypsum helps to retard the setting time of cement when mixed with water and, hence, gives sufficient time for transportation and placement of the wet concrete mixture.

Figure 8.3 Manufacturing of portland cement by dry process



(Courtesy of R. Tuladhar, James Cook University, Australia)

Clinkers

Figure 8.4

(Courtesy of D. Moore, www.cementkilns.co.uk)

Chemical Composition

As discussed earlier, the main raw materials used in production of cement are limestone, clay, shale, and iron ore. Limestone ($CaCO_3$), when burned at a high temperature in a kiln, produces lime (calcium oxide, CaO) and carbon dioxide (CO_2). This reaction is called calcination. On the other hand, clay, shale, and iron ore produce silica (SiO_2), alumina (Al_2O_3), iron oxide (Fe_2O_3), and water. During the clinker-forming process, these chemicals react to form four principal compounds—tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrite (C_4AF) (Table 8.1), which together constitute cement.

Hydration of Cement

When cement is mixed with water, the constituent compounds of cement chemically react with water to form a hardened paste. This reaction is called *hydration of cement*, and it results in rapid strength development. Heat generated during hydration of cement due to the exothermic chemical reaction (reaction that generates heat) between cement and water is called the *heat of hydration*. Figure 8.5 shows the rate of hydration in terms of heat of hydration evolved as a function of time. Stage I occurs immediately after the cement is mixed with water. A large amount of heat is

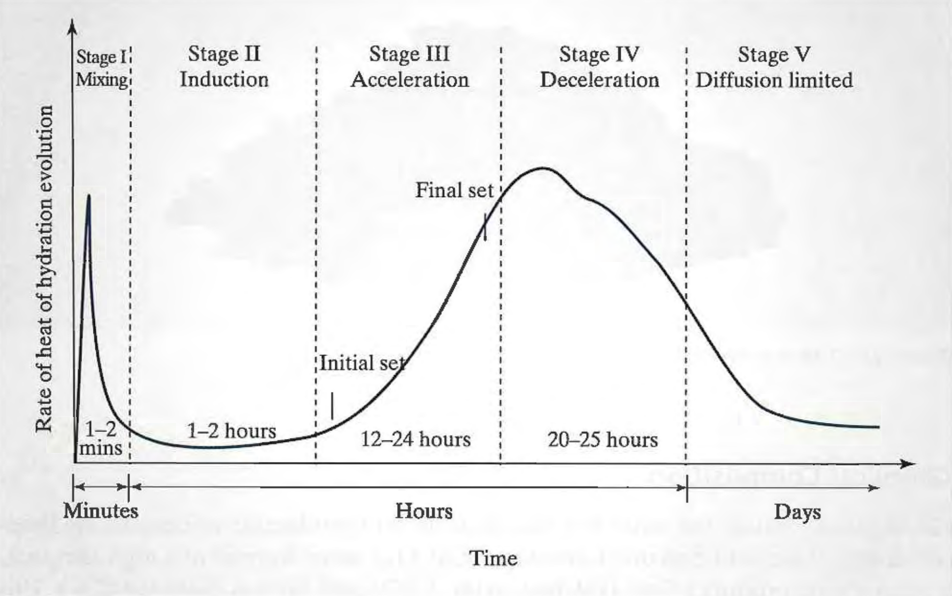
Principal chemical compounds that constitute cement

Table 8.1

Compound	Chemical formula	Abbreviation*	Percentage by weight (%)	Heat of hydration (J/g)
Tricalcium silicate	$3CaO.SiO_2$	C_3S	45–65	500
Dicalcium silicate	$2CaO.SiO_2$	C_2S	10–30	250
Tricalcium aluminate	$3CaO.Al_2O_3$	C_3A	6–12	850
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	C_4AF	6–8	400

* *where* $CaO = C$, $SiO_2 = S$, $Al_2O_3 = A$, *and* $Fe_2O_3 = F$

Figure 8.5 Rate of cement hydration and heat of hydration evolution over time



(Courtesy R. Tuladhar, James Cook University, Australia)

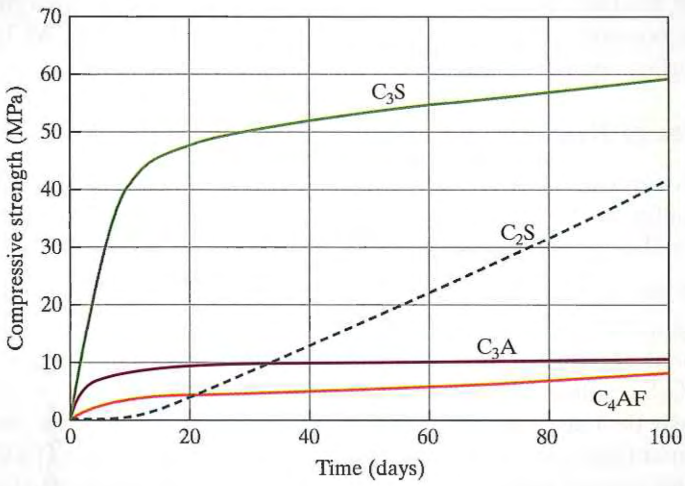
generated due to the initial rapid hydration of cement. This stage lasts for a very short duration (in the order of minutes). Stage II is called an induction period or dormancy period when the hydration reaction and heat generation slows down significantly. Concrete is fluid during this period (1–2 hours long), and it needs to be properly mixed, transported, placed, and finished during this plastic state. During Stage III, the acceleration or hardening period, C_3S compound in cement is hydrated rapidly. The concrete starts to harden and develop mechanical stiffness and strength during this stage. Stage IV, the deceleration or cooling period, is marked by the decrease in hydration of cement due to the layers of hydration product covering cement particles. Stage V is called the diffusion limited stage where the hydration reaction occurs at a very slow rate slow as the water and cement particles need to diffuse in order to react. This stage may continue for years and the concrete densifies throughout the stage.

The increase in temperature of concrete due to the heat of hydration depends on the type of cement used, volume of concrete cast, rate of placement of concrete, the ambient temperature, and the temperature of the concrete during casting. For massive concrete constructions like dams, retaining walls, and large abutments, the heat of hydration can be significantly high with a tendency to cause thermal cracking. Supplementary cementitious materials like *ground granulated blast furnace slag* (GGBS), *fly ash*, or natural pozzolans are often used to partially replace cement in concrete to slow down both the heat and the rate of hydration. Use of supplementary cementitious material in concrete is discussed further in Section 8.2.2.

Ordinary portland cement consists of 45 to 65 percent tricalcium silicate (C_3S). Within few hours of mixing cement and water, C_3S starts to hydrate, generating a large amount of heat. The rapid hydration of C_3S is mainly responsible for the early setting and early age strength of concrete (Figure 8.6). Dicalcium silicate (C_2S) constitutes about 10 to 30 percent of cement. C_2S hydrates slowly with less evolution of heat. It is responsible for strength gain at later ages of concrete.

Development of strength of compounds in portland cement on hydration

Figure 8.6



(Courtesy R. Tuladhar, James Cook University, Australia)

Tricalcium aluminate (C_3A) and tetracalcium aluminoferrite (C_4AF), formed from alumina and iron ore, respectively, act as fluxing agents and help to control the temperature of clinkers and facilitate lime and silica reaction. C_3A reacts very rapidly with water and tends to cause immediate setting of the cement paste, releasing considerable amount of heat. Gypsum present in cement helps to retard the rapid hydration of C_3A and prevents this *flash setting*. C_3A does not contribute much toward the long-term strength of concrete and is also susceptible to sulphate attacks. C_4AF also has little contribution to the strength of the cement paste. The distinct grey color of cement is due to the presence of C_4AF.

C_3S and C_2S, when hydrated, produce calcium silicate hydrate ($3CaO.2SiO_2.4H_2O$) or simply written as C-S-H) and calcium hydroxide ($Ca(OH)_2$). C-S-H is a hydration product that consists of rigid needle-like crystals (known as ettringite) growing from the surface of the unhydrated cement particles. It gives strength to the hardened cement paste. $Ca(OH)_2$ is a byproduct of hydration reaction and does not contribute to the strength development. It, however, raises the pH of concrete pore water up to 12.5.

$$\underset{\text{Calcium silicates}}{\begin{matrix}C_3S\\C_2S\end{matrix}} + \underset{\text{Water}}{H_2O} \xrightarrow{\text{Hydration}} \underset{\text{Calcium silicate hydrates}}{\text{C-S-H}} + \underset{\text{Byproduct}}{Ca(OH)_2}$$

Physical Properties

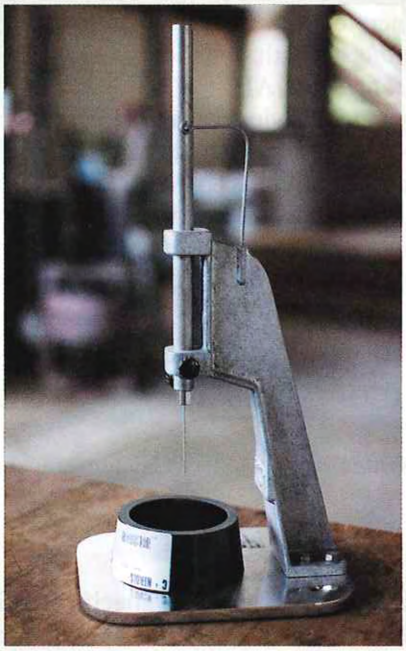
Portland cement is a fine grey powder with specific gravity of 3.15 and is determined according to ASTM C188 (2009). The fineness of cement is usually represented in terms of specific surface area (SSA). The Blaine method (ASTMC204, 2011) or the Wagner turbidimeter apparatus (ASTM C115, 2010) can be used to determine specific surface area of cement. Ordinary portland cement (ASTM Type I) has fineness of 300 to 400 m^2/kg. Cement with higher fineness has larger surface area, and hence, also has the higher rate of hydration and higher rate of strength gain. Rapid-hardening portland cement or high-early strength cement (ASTM Type III) has a specific surface area of 450 to 600 m^2/kg (2200 to 2930 ft^2/lb).

Properties of Hydrated Cement Paste

Chemical composition of cement and the additional materials blended with cement have a significant effect on the properties of the hydrated cement paste, such as setting time and compressive strength.

Setting Time With the progression of hydration reaction, cement paste gains stiffness and gradually changes from a fluid to a rigid state. This is called *setting*. The development of strength in the set cement paste is called *hardening*. Hydration of C_3A and C_3S results in the initial set of cement paste. Electrical conductivity of the cement paste decreases after the final set. Consistency and the initial and final setting time of cement paste are determined by the Vicat needle test (ASTM C191, 2008) on sample cement paste with *normal consistency*. Water required to achieve normal consistency is also determined using Vicat apparatus (Figure 8.7). Cement with normal consistency is the one where a 10 mm (0.4 in.) diameter plunger penetrates to 4–6 mm (0.16–0.24 in.) from the bottom of the mold. The initial set time is the time elapsed from the time of mixing cement with water to the time when a needle of diameter

Figure 8.7 Vicat apparatus to determine setting time of cement



(Courtesy of R. Tuladhar, James Cook University, Australia)

1 mm penetrates 25 mm (1.0 in.) or less into the paste. The cement is considered to be finally set when the needle does not sink visibly into the paste.

Compressive Strength Cement paste develops strength from the hydration reaction. The hardening cement paste binds the aggregates and other materials to form the hardened concrete. The mechanical strength is, hence, a very important property of the hardened cement paste. Strength tests are performed on cement–sand mortar. According to ASTM C109/C109M (2011a), a compression test is carried out on 50 mm (2.0 in.) mortar cubes prepared with 1:2.75 cement-sand mortar with water/cement ratio of 0.485.

8.2.2 Supplementary Cementitious Material

Concrete industries these days use a wide range of supplementary cementitious material (SCM), as a partial replacement for portland cement. The portland cement production process is an energy-intensive process. Moreover, burning fossil fuel in rotary kilns and calcination reaction during the formation of clinkers produces a large amount of carbon dioxide. It is estimated that production of 1 tonne of cement produces approximately 1 tonne of carbon dioxide. Use of alternative cementitious material to partially substitute portland cement in concrete reduces the consumption of portland cement and thus reduces the carbon footprint of concrete. Commonly used supplementary cementitious materials such as fly ash, ground granulated blast furnace slag, and silica fume are obtained as a byproduct of other industrial plants. Apart from these industrial byproducts, naturally occurring pozzolanic materials such as volcanic ash, opaline shale, pumicite, and tuff can also be used as partial cement replacements. Pozzolans are siliceous or aluminosiliceous materials that react chemically with calcium hydroxide (byproduct of hydration reaction) in the presence of moisture to form calcium silicate hydrates that are responsible for the strength development in concrete.

$$\underset{\text{Calcium silicates}}{\begin{matrix} C_3S \\ C_2S \end{matrix}} + \underset{\text{Water}}{H_2O} \xrightarrow{\text{Hydration}} \underset{\text{Calcium silicate hydrates}}{C\text{–}S\text{–}H} + \underset{\text{Byproduct}}{Ca(OH)_2}$$

$$\underset{\begin{matrix}\text{Byproduct of cement}\\ \text{hydration}\end{matrix}}{Ca(OH)_2} + \underset{\begin{matrix}\text{(Reactive silica or}\\ \text{alumina compounds)}\end{matrix}}{\text{Pozzolan}} \xrightarrow[\text{Pozzolanic reaction}]{\text{Moisture}} \underset{\text{Calcium silicate hydrates}}{C\text{-}S\text{-}H}$$

Commonly Used SCMs

Some commonly used SCMs are discussed briefly in this section. This includes industrial byproducts and some natural ones.

Fly Ash Fly ash, also known as pulverized-fuel ash, is a byproduct of coal-fired thermal power plants. Fly ash (Figure 8.8 a) particles are spherical with size ranging from 0.5 to 100 μm (ACI 232.R, 2004). Fineness of fly ash ranges from 250 to 600 m^2/kg (1220 to 2930 ft^2/lb). Fly ash primarily contains silica, alumina, iron oxide, and lime. ASTM C618 (2008a) classifies fly ash into two categories: *Class F* and *Class C*.

Figure 8.8 Supplementary cementitious materials (SCM) (a) Fly ash (b) Ground granulated blast furnace slag (c) Silica fume

(a) (b) (c)

(Courtesy of R. Tuladhar, James Cook University, Australia)

Class F fly ash is produced from anthracite and bituminous coal and contains less than 5–10% lime (CaO). Class F fly ash is essentially pozzolanic material, meaning it does not react with water on its own, but it reacts with $Ca(OH)_2$, a byproduct of cement hydration reaction, in the presence of moisture to give calcium silicate hydrates.

Class C fly ash is typically derived from lignite and subbituminous coal and contains a high amount of CaO (15–30%). Apart from its pozzolanic property, Class C fly ash has a hydraulic (cementitious) property as well. It can react with water on its own to form calcium silicate hydrates.

Fly ash is widely used in concrete mixtures these days as a partial replacement of portland cement. Class F and Class C fly ash are generally used at dosage of 15–25 percent and 15–40 percent, respectively, by mass of cementitious material. Use of fly ash as SCM in concrete reduces the permeability, and hence, improves the durability and strength of concrete. The spherical shape of fly ash particles also improves the workability and cohesiveness of concrete. Fly ash also reduces the rate of hydration of cement and lowers the heat of hydration, which is particularly beneficial where a large mass of concrete is poured. Use of fly ash in concrete can lower the rate of strength gain at early age. However, the pozzolanic reaction of fly ash continues for a very long period, improving the ultimate strength of the concrete.

Ground Granulated Blast Furnace Slag Slag is a byproduct of iron industries. Slag, obtained from iron industries, when cooled and ground to the fineness of cement (generally less than 45 microns) is called ground granulated blast furnace slag (GGBS) (Figure 8.8b). Portland cement combined with GGBS is called portland blast furnace cement or *slag cement*. GGBS has hydraulic properties. GGBS in portland blast furnace cement is activated by alkali hydroxides (NaOH and $Ca(OH)_2$) produced from Portland cement hydration, and it reacts with water to form calcium silicate hydrates. GGBS is used at 20–70 percent by mass of the cementitious materials. Slag cement with high volume of GGBS is used as low-heat cement in mass concrete constructions where the heat of hydration is critical. Furthermore, the use of GGBS with portland cement improves workability, long-term strength, and durability. ASTM C 989 grades blast furnace slag into Grades 80, 100, and 120 according to its hydraulic activity, with the higher grade contributing more to strength potential.

Silica Fume Silica fume (Figure 8.8c) particles are spherical in shape and are extremely fine (with average diameter of 0.1μm) and contains 90 percent reactive silica. It is collected as a byproduct from electric arc furnace in silicon and ferrosilicon industries. The extremely small size of silica fume makes it a highly reactive pozzolanic material that reacts with $Ca(OH)_2$ to form calcium silicate hydrates. It also increases the strength of concrete due to its extreme fineness, which helps to achieve better packing in concrete. It is, therefore, widely used in producing high-strength concrete. Silica fume is usually used in the range of 5–10 percent of the total mass of the cementitious material.

Silica fume is often used to produce high-strength concrete or when significantly reduced permeability is required. However, it should be considered that the cost of silica fume is significantly higher than the cost of fly ash or slag, and due to its extreme fineness, special procedures for handling, placing, and curing are required when using silica fume in concrete.

Natural Pozzolans Apart from the industrial byproducts, naturally occurring pozzolanic materials such as volcanic ash, opaline shale, pumicite, and tuff can also be used as partial cement replacement. The term *pozzolan* is derived from volcanic ash from the place called Pozzuoli in Italy, and its use in producing water-resistant construction material dates back to the Romans in 200 BCE. In present days, metakaolin, calcined shale, or calcined clay are commercially available natural pozzolans and are produced by controlled firing of naturally occurring minerals. Other natural pozzolans include rice husk ash, diatomaceous earth, volcanic glass, zeolite trass, or tuffs. ASTM C 618(2008a) classifies natural pozzolans as Class N.

Effect of Supplementary Cementitious Materials on Properties of Fresh and Hardened Concrete Using SCMs in concrete is one of the effective ways of producing sustainable concrete structures. SCMs in concrete can enhance both fresh and hardened properties of concrete. In addition to the improvement in durability, workability, and strength of concrete, use of SCMs can have direct environmental benefits as well. Environmental, economic, and technical benefits of using SCMs in concrete are explained below:

a. **Environmental benefits:** SCMs play a critical role in lowering the carbon footprint of concrete. Manufacturing of portland cement, a chief binding material in concrete, is an energy-intensive process and accounts for 5 percent of total CO_2 emission from human activities. Replacement of portland cement by SCMs results in lower consumption of portland cement; and decreases energy consumption and CO_2 emission. Furthermore, SCMs are generally byproducts from other industries. Their use in concrete will reduce their disposal into the environment and the associated environmental impacts.
b. **Economic benefits:** Portland cement is the most expensive ingredient in concrete; and therefore, replacement of portland cement with SCMs also has direct economic advantage apart from the obvious environmental benefits. In addition to this, use of SCMs will also reduce the cost of land filling these industry byproducts.

c. **Technical benefits:** In general, use of SCMs in concrete improves consistency and workability of fresh concrete, due to additional volume of fines in the mixture. Fly ash and GGBC generally improve workability of concrete. Use of silica fume can, however, reduce the workability of concrete. Superplasticizers are often used with silica fume to enhance workability of silica fume concrete.

SCMs when used at higher percentages can slower the rate of hydration in concrete, which retards its setting time and also reduces heat of hydration. Retardation of setting time of concrete is particularly beneficial in hot weather. Reduction of heat of hydration is beneficial in mass concrete applications where excessive heat evolution during casting can cause thermal cracking in concrete. SCMs can also reduce excessive bleeding in concrete and prevent plastic settlement cracking in concrete. Silica fume is especially very effective in reducing bleeding in concrete.

Use of SCMs improves the long-term strength of concrete. SCMs, such as fly ash and GGBS, may slow the rate of initial strength gain in concrete, but strength gain continues for a longer period in SCM concrete mix compared to portland cement–only mix. SCMs generally result in higher ultimate strength. Use of high-strength concrete allows more efficient design of concrete sections in large concrete structures like high-rise buildings and long-span bridges. Ternary blended concrete with portland cement (50–60%), fly ash (25–50%) and silica fume (5–10%) was used in construction of Burj Dubai Building in Dubai for achieving a high level of workability, durability, and strength. Blended cements are also used in backfilling large underground mine voids, where the mine tailings are mixed with small dosages of blended cements or Portland cement.

Furthermore, the fineness of SCMs and reduction in capillary pores due to its reaction with cement hydration products also contributes in reducing permeability of concrete. Reduction in permeability of concrete improves its resistance to chloride-ion penetration and sulphate attacks, and significantly improves the durability of concrete structures. Use of SCMs in concrete is also found to mitigate alkali–silica reaction.

8.2.3 Water

Another important ingredient of concrete mixture is water. Already seen in the preceding sections, chemical reactions between cement and water in the concrete mixture are responsible for the strength development in concrete. Water also makes concrete sufficiently workable. Both the quantity and quality of water used can significantly affect workability and strength of the resulting concrete. Sufficient water should be used to ensure the hydration reaction; on the other hand, an excessive amount of water will compromise the strength of concrete.

Water used in mixing concrete should be free from impurities like chlorides, sulphates, carbonates, organic acids, organic matter, or other suspended solids. Excessive amount of impurities in water can affect setting time and concrete strength. An excessive amount of chloride in water can cause corrosion of reinforcements embedded in concrete structures. Chlorides may also cause white staining on the concrete surface (known as efflorescence) due to formation of calcium carbonate. It is for this reason that sea water is not used in concrete mixes for reinforced concrete constructions. Sulfates in water can affect the setting time and strength of the concrete. Carbonates can accelerate the setting time, whereas organic acids retard the setting time of the concrete. Large amounts of suspended solids like clay and silt in the water can significantly reduce the durability and strength of the concrete.

ASTM C1602 (2006) prescribes limits on sulphates, chlorides, and suspended solids in mix water. Generally speaking, if the water is drinkable (potable), it is suitable for mixing concrete.

Waste wash water collected at ready-mix plants is now increasingly being used in concrete mixing. Waste water from concrete plants is highly alkaline and should not be discharged into rivers and waterways as it may have adverse effects on aquatic life. Instead, waste wash water in ready-mix plants can be recycled into concrete. ASTM C94 (2011b) has permitted the use of waste wash water in concrete. However, caution should be taken, as high alkalinity and excessive amount of suspended solids in the reclaimed water can negatively affect setting time and increase water demand in concrete.

Water/Cement Ratio

The *water-cement (w/c) ratio* is one of the crucial factors that affect the strength of concrete. The w/c ratio is defined as the ratio of the total water available in the mix and the weight of cementitious materials. Apart from the water directly added to the mix, it is important to take into account the extra water on the surface of the wet aggregates. If the aggregates are dry, water is absorbed by the aggregates and should be accounted for. W/c ratio is calculated as:

$$w/c\ ratio = w/W_c \tag{8.1}$$

where w is the weight of water in the mix, considering that the aggregates are in surface saturated dry (SSD) condition (Explained in Section 8.7); and W_C is the total weight of cementitious materials.

An increase in w/c ratio decreases the strength of concrete and vice versa. However, sufficient water should be used in the mix to ensure the proper hydration of cement and acceptable workability. Generally, the least amount of w/c ratio which gives acceptable workability is used for all construction.

Water demand is the amount of water required to provide adequate workability to a cubic meter of concrete. It is significantly influenced by grading of the aggregates, type of cement used, texture and size of coarse aggregates, and the use of *admixtures*. Water-reducing admixture can be used in concrete to reduce the water demand while retaining the workability of the mix.

8.2.4 Aggregates

Aggregates constitute 70 to 80 percent by volume of concrete and contribute to dimensional stability, chemical and abrasion resistance, and strength of the concrete. They range from coarse stone particles to fine sand. Aggregates in concrete also bring thermal compatibility between concrete and embedded steel in reinforced concrete structures. Aggregates are significantly cheaper than cement, and hence they help to reduce the cost of the concrete.

Natural aggregates (sand and gravel) or manufactured aggregates (crushed sand and stone) can be used in the concrete. The type, size, and texture of aggregates affect the water demand, strength, and durability of concrete. Classification and physical properties of aggregates in general are discussed in Chapter 5. This section mainly focuses on the use of aggregates in concrete.

Fine Aggregates

Fine aggregate or sand is less than 4.75 mm in size (No. 4 ASTM sieve). Use of fine aggregates improves the workability, strength, and durability of concrete. The *Fineness modulus* of good-quality sand ranges between 2 and 3. The fineness modulus (FM) is a measure of the average fineness or coarseness of aggregate and is defined as 1/100 of the cumulative percentages retained on the sieves of standard series 150 μm, 300 μm, 600 μm, 1.18 mm, 2.36 mm, and 4.75 mm. The use of very fine sand can increase the water demand. ASTM C33 (2011a) limits the amount of material passing through No. 200 sieve to 3 percent.

$$FM = \frac{\Sigma(Cumulative\ \%\ retained\ on\ standard\ sieve \geq 150\ \mu m)}{100} \qquad \textbf{(8.2)}$$

In a special type of concrete known as *pervious concrete*, no fines or a very small amount of fines (around 7%) is used (ACI 522R, 2010). Use of no sand or a small amount of sand in pervious concrete results in larger voids and ensures that water can percolate through it. These days, pervious concrete is getting popular for use in car parks, driveways, and footpaths as it allows storm water to penetrate into the ground, recharging the ground water. However, the use of no sand or a limited amount of sand compromises the strength of pervious concrete.

EXAMPLE 8.1

Sieve analysis data of a fine aggregate sample used for a concrete mix is shown in the following table. Calculate the fineness modulus for the given fine aggregate. Plot the grading curve for the given fine aggregate and compare it with the fine aggregate grading range specified in ASTM C33 (2011a) in Figure 8.11.

Solution:

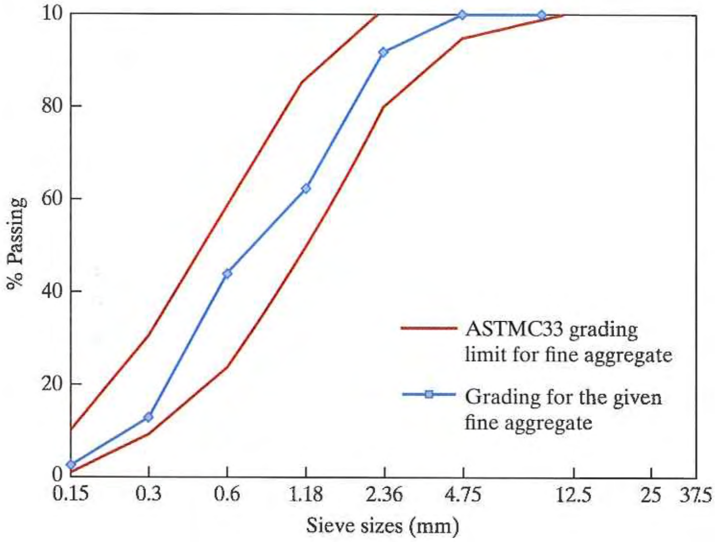
Total cumulative percentage retained and cumulative percentage passed, by mass, is calculated as shown in the table.

ASTM Sieve No.	Sieve size	Mass retained (g)	Percentage retained (%)	Cumulative percentage retained (%)	Cumulative percentage passing (%)
3/8 in.	9.5 mm	0	0.0	0.0	100.0
4	4.75 mm	0	0.0	0.0	100.0
8	2.36 mm	30	7.8	7.8	92.2
16	1.18 mm	115	29.9	37.7	62.3
30	600 μm	70	18.2	55.9	44.1
50	300 μm	120	31.3	87.2	12.8
100	150 μm	39	10.2	97.4	2.6
Pan		10	2.6	—	—
		Total = 384g		Total = 286.0	

Fineness modulus is given by Equation 8.2 as:

$$FM = \frac{\Sigma(Cumulative\ \%\ retained\ on\ standard\ sieve \geq 150\ \mu m)}{100} = \frac{286.0}{100} = 2.86$$

The grading for the given fine aggregate sample is shown along with the grading limit for fine aggregate as specified by ASTM C33 (2011a).



Coarse Aggregates

Coarse aggregates have a size greater than 4.75mm, and they provide dimensional stability to the hardened concrete. In general, aggregates in concrete need to be strong and hard, durable, chemically inert, and clean and free from any impurities. The fineness modulus of coarse aggregate is 5.5 to 8. Coarse aggregates for concrete can be categorized into four types (Table 8.2), according to their relative density and strength. *Relative density* of coarse aggregates for normal weight concrete ranges from 2.65–2.75.

Particle Shape and Texture Aggregates can be of different shapes, sizes, and texture. As explained in Chapter 5, coarse aggregate shapes can vary from round, angular, elongated, to flaky (Figure 8.9). Round aggregates have lower water demand compared to the angular aggregates. However, angular aggregates bond better with the cement paste. Elongated and flaky aggregates in excess of 10 to 15 percent of mass of coarse aggregate are not desirable as components of concrete, as they adversely affect the durability and strength. Surface texture of aggregates also influence its bond with cement paste. Aggregates with coarse surfaces bond better with the cement paste; however, the coarse surface may reduce the workability of the concrete.

Table 8.2 Classification of coarse aggregate

Class and density of aggregates	Examples	Uses	Resulting concrete density kg/m³ (lb/ft³)
Heavy weight (> 3200 kg / m³)	Barytes	Heavy weight concrete structures, nuclear plants	3000–5000 (187–312)
Normal weight (> 2100 kg / m³)	Natural sands, gravels, rock	Normal weight concrete structures	2000–2600 (125–162)
Lightweight (>2100 kg / m³)	Foamed iron blast-furnace slag	Lightweight concrete structures	1000–1500 (62–94)
Ultra lightweight (> 500 kg / m³)	Perlite	Thermal insulation	500–1000 (31–62)

Note: Values in parentheses are in lb/ft³

Figure 8.9 Different shapes and sizes of aggregates: (a) 10 mm angular (b) 20 mm angular (c) 14 mm round

(a) (b) (c)

(Courtesy of S. Nash, Department of Transport and Main Roads, Queensland Australia)

Grading of Aggregates

Grading of aggregates has significant influence on the workability of the concrete mix. Sand is usually well graded, with grains from fine to coarse sizes to ensure good workability and packing of materials in the concrete to give better strength performance. Coarse aggregate, on the other hand, can be graded or single size, depending upon the type of concrete required.

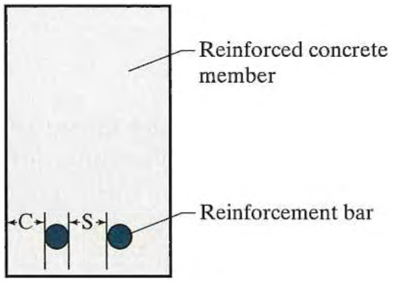
Aggregates with larger maximum size have lower surface area to cover by the cement paste, and hence have lower water demand and better workability. The maximum size of coarse aggregate in concrete is governed by the spacing between the reinforcing bars and the dimensions of the structure and formwork. According to ACI 318 (2011), the maximum size of coarse aggregate should not exceed:

a. 1/5 of the narrowest dimension between sides of forms
b. 3/4 of the clear space between individual reinforcing bars (S in Figure 8.10)
c. 3/4 of the clear space between reinforcement and forms (C in Figure 8.10)
d. 1/3 of the slab thickness for unreinforced slabs

Figure 8.11 shows the desirable grading zones for fine and commonly used coarse aggregates specified in ASTM C33 (2011a). Grading of coarse aggregate has significant influence on workability of concrete—a well-graded coarse aggregate will result in better workability of concrete than a gap-graded aggregate.

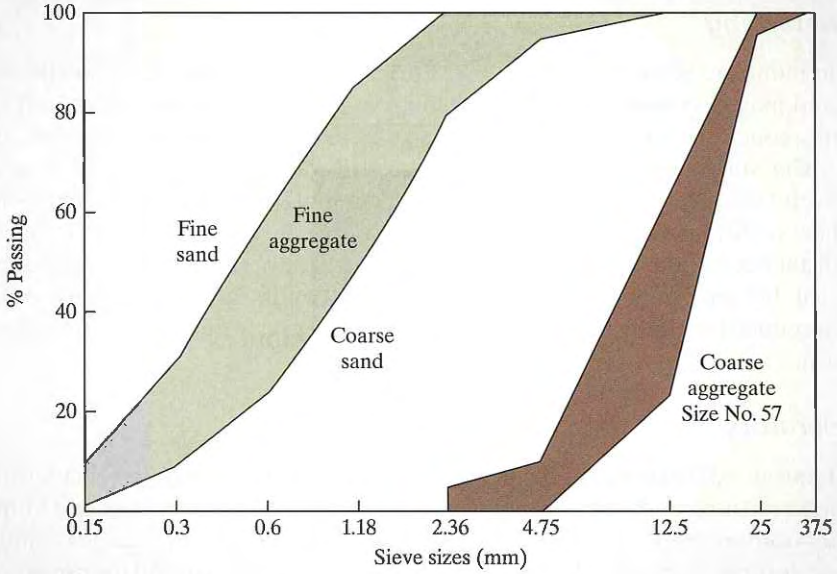
Maximum size of coarse aggregate criteria

Figure 8.10



ASTM C33 specifications for grading of fine aggregate and for commonly used coarse aggregate

Figure 8.11



(Based on ASTM C33 (2011a), "Specification for Concrete Aggregates," ASTM International, West Conshohocken, PA)

The shape, maximum size, and amount of coarse aggregate affect the amount of mixing water required in concrete to maintain the given slump. The larger the size of coarse aggregate, the lower the surface area required to be wetted is and hence reduces the amount of water required to achieve required workability (Table 8.11). Rounded aggregates also result in better workability compared to the crushed stones, which are angular in shape.

8.2.5 Admixture

Admixtures are the materials added to concrete to modify particular properties of fresh or hardened concrete. Admixtures are added just before or during the mixing of concrete, at the batching plant or at the site. They are usually added as solutions so that they can easily be mixed with concrete. ASTM C494 (2011) classifies admixtures, according to their functions, in the following basic categories: air-entraining, set-retarding, accelerators, water reducers, and superplasticizers. These are described below.

Air-Entraining

Air-entraining agents create minute air bubbles (about 10 μm to 1000 μm in diameter), distributed throughout the concrete. These tiny air-filled voids give space for expansion of water during the freezing and thawing cycles. Entrained air bubbles are, however, different than the accidentally-entrapped large air bubbles in the concrete. Air-entraining agents also help to minimize bleeding and segregation and improve workability in concrete. However, due to the increase in voids, addition of air-entraining agents may reduce the strength of concrete (see Figure 8.25). Examples of air-entraining agents are salts of fatty acids derived from animal and vegetable fats and oils, alkali salts of wood resins, and alkali salts of sulphate and sulfonated organic compounds.

Set-Retarding

As the name suggests, retarding admixtures (*retarders*) cause delay in the setting time and may also slow down the hardening of the cement paste. In hot and windy climates, concrete can set very quickly. Use of retarders will help to delay the setting time, giving sufficient time to transport, place, and compact the concrete. It is particularly useful to avoid cold joints during construction of large concrete structures in hot weather conditions. Retarders are also added when concrete needs to be transported long distances. Examples of commonly used retarders are sugar, carbohydrate derivatives, soluble zinc salts, soluble borates, and methanol. Early strength of concrete can be reduced while using retarders. Retarding agents often have water-reducing properties as well.

Accelerators

Accelerating admixtures, or accelerators, are added in concrete to accelerate the hydration reaction and early strength gain in concrete. It, however, does not influence the long-term strength of concrete. Accelerators are used in cold weather conditions or when formworks need to be removed quickly. They are also useful for urgent repair works. In hot weather or mass concrete constructions, accelerators can significantly increase the heat of hydration, causing undesirable effects like shrinkage cracks in concrete.

Calcium chloride and sodium chloride are examples of accelerators that decrease the setting time and increase the rate of strength development in concrete. However, the use of calcium chloride can be detrimental as the chloride ions in concrete can cause corrosion of the embedded reinforcement (Figure 8.12); furthermore, it reduces the resistance of concrete to sulphate attacks. For these reasons, calcium chloride should never be used as an accelerator in reinforced concrete, prestressed concrete, or even in plain concrete structures that are susceptible to alkali–aggregate reaction and sulphate attacks. Alternatively, calcium nitrite, calcium nitrate, calcium formate, and sodium formate can be used as accelerators in such conditions.

Water Reducers

Water-reducing admixtures, also known as plasticizers, reduce the water/cement ratio while retaining the workability of the concrete. If the water/cement ratio is specified from the strength requirement, addition of water-reducing admixtures will improve the workability of the concrete at the constant water/cement ratio.

Use of water-reducing admixtures can reduce the required water content of the mix by 5 to 10 percent; hence, they increase the strength of the concrete. Concrete with specified low water/cement ratio often uses water-reducers to achieve sufficient workability. They are also used when concrete with high strength and high workability is required. Lignosulfonic acids and their salts and hydroxylated carboxylic acids and their salts are generally used as water-reducing admixtures. These admixtures also delays the setting time of concrete. They also help to prevent segregation in concrete.

Figure 8.12

Corrosion of reinforcement caused by the use of calcium chloride as an accelerator

(Courtesy of A. Vanderstaay, Department of Transport and Main Roads, Queensland Australia)

Superplasticizers

At a constant water/cement ratio, the addition of superplasticizers considerably improves the workability of concrete, producing "flowing concrete." Concrete with superplasticizers can be placed in difficult locations, such as inaccessible areas and where reinforcements converge resulting congestion.

On the other hand, superplasticizers are also used to produce extremely high-strength concrete with normal workability. Use of superplastizers can reduce the water content by 25 to 35 percent, keeping the same workability. Extremely low water/cement ratio will result in very high-strength concrete. Commonly used superplasticizers are sulfonated melamine formaldehyde condensates, and sulfonated naphthalene formaldehyde condensates. The increase in workability with addition of a superplasticizer lasts only for 30–60 minutes; hence, it is important to place the concrete as soon as possible after its addition in the concrete mix. Overdoses of superplasticizers can lead to extreme segregation of concrete; hence, their dosage should be precisely controlled.

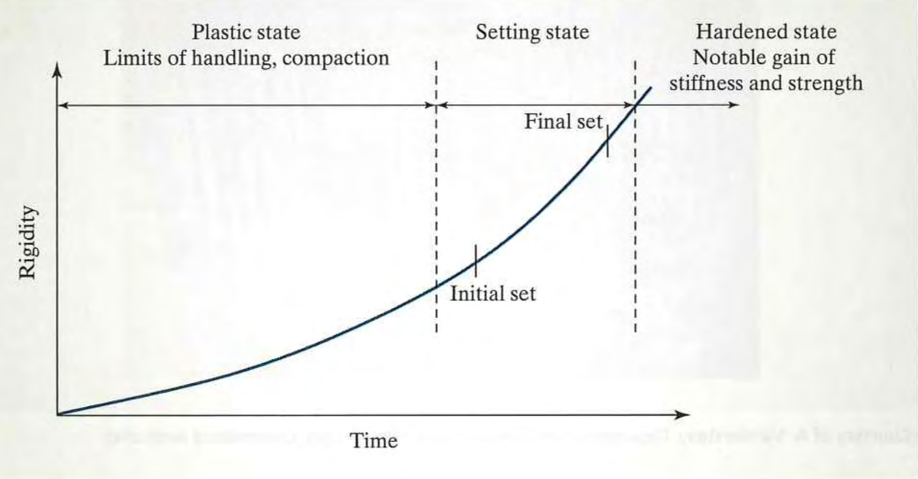
8.3 Different Stages of Concrete

Three different stages of concrete are shown in Figure 8.13.

8.3.1 Plastic State

Concrete is fluid and soft just after mixing. It can be worked and molded into different shapes. Concrete at this state is called plastic state concrete or simply fresh concrete. The plastic state of concrete lasts only for few hours, after which it starts to stiffen and gain strength. Concrete needs to be properly mixed, transported, placed, and compacted when it is still in the plastic state.

Figure 8.13 Different stages of concrete



8.3.2 Setting State

After concrete is being placed and compacted, it starts to stiffen due to formation of early stage calcium silicate hydrates. Concrete at this stage is no longer fluid. However, at this stage the concrete is still weak as it has not achieved its full strength.

Elapsed time from the mixing of concrete to the time when it becomes stiff and cannot be vibrated without damaging the concrete is called initial setting time. Final setting time is referred to as the point when the concrete starts to develop mechanical stiffness and strength. In a hot climate, the setting time of concrete is considerably reduced. Set-retarders can be used in hot weather conditions to get sufficient time for transporting, placing, compacting, and finishing the concrete. On the other hand, in the low-temperature conditions, setting time is considerably delayed. Accelerators can be used to reduce the setting time of concrete in cold weather conditions.

8.3.3 Hardening State

Concrete gains mechanical strength and stiffness notably during the hardening state. The proceeding hydration of cement contributes toward the development of strength of concrete with time. Concrete continues to gain strength for weeks, months, and even years. The rate of strength gain is rapid at the initial stage but decreases in the later weeks.

8.4 Properties of Fresh Concrete

Concrete needs to be in the plastic state during transportation, placement, compaction, and finishing. Important properties of plastic concrete are workability, consistency, and cohesiveness. The properties of fresh concrete also affect the long-term strength, durability, and other properties of hardened concrete.

8.4.1 Workability

Workability is the ease with which the concrete ingredients can be mixed, transported, placed, compacted, and finished with minimum homogeneity loss (ASTM C125, 2011a). Workability is essentially a property of fresh concrete; however, it has significant influence on the properties of hardened concrete. Sufficient workability in concrete ensures proper compaction and prevents segregation of concrete resulting in a strong and durable concrete. Workability cannot be directly measured; nevertheless, it is a very important property of concrete. It is assessed on site by concrete layers in terms of the effort required in handling the concrete mix.

Factors Affecting Workability

Increasing water content in concrete (i.e., increase in w/c ratio) increases the ease in handling of concrete. However, an increase in the w/c ratio compromises the strength of the concrete and also can lead to segregation. Alternatively, workability can also be improved at constant w/c ratio by adjusting aggregate/cement ratio, aggregate type, aggregate grading, and admixtures.

For a given water/cement ratio, reducing the aggregate/cement ratio increases the workability. As discussed in preceding sections, grading of aggregate also has significant influence on workability of the concrete mix—well-graded fine aggregate gives better workability. A small amount of fine aggregate can lead to a mix that is prone to segregation and difficult to finish. On the other hand, an excessive amount of fine aggregate results in sticky concrete

Workability can also be improved by increasing maximum coarse aggregate size in the mix. The larger the maximum size of coarse aggregate, the smaller the surface area required to be wetted per unit mass. Moreover, aggregate texture and shape also has a substantial influence on the workability of concrete. Smooth texture aggregates improve workability. Similarly, round aggregates result in more workable concrete than angular shape aggregates. Increase in the ambient temperature can be detrimental to the workability of concrete. In hot weather conditions, concrete is often mixed and placed at night to avoid the extreme heat. As already discussed in Section 8.2.5, chemical admixtures such as water-reducers and superplasticizers are often used to improve workability in concrete with low water/cement ratio. Air-entraining admixtures also improve workability of concrete owing to the spherical shape of fine air bubbles entrained into the concrete.

8.4.2 Consistency

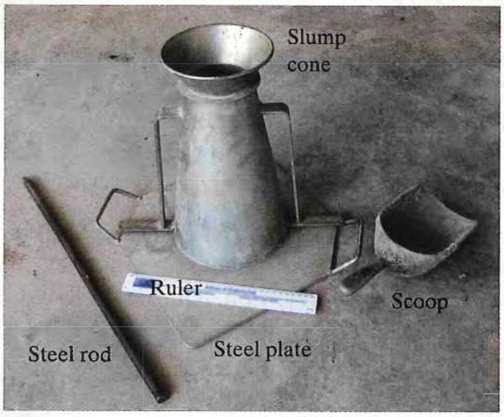
Consistency of fresh concrete is defined as its relative mobility or ability to flow (ACI 116R, 2000). In other words, consistency measures the fluidity of the concrete.

Slump Test

Slump tests are widely used in the field and also in the laboratory to measure consistency of concrete. In addition, slump tests can be readily done to test batch-to-batch consistency of the supplied concrete at the site. Too high or too low a *slump* gives an immediate warning of inconsistency in the supplied concrete.

Tools required for the slump test (Figure 8.14) are (1) standard slump cone [100 mm (4 in.) top diameter × 200 mm (8 in.) bottom diameter × 300 mm (12 in.) high frustum cone]; (2) bullet-nose steel rod of 16 mm (0.6 in.) diameter; (3) slump plate [500 mm (20 in.) × 500 mm (20 in.)]; (4) ruler; and (5) scoop. Slump tests are conducted according to ASTM C143 (2010a).

Figure 8.14 Slump test apparatus



(Courtesy of R. Tuladhar, James Cook University, Australia)

The internal surface of the cone is moistened and is placed on a clean, leveled slump pate. The mold is firmly held on its base and is filled with fresh concrete in three layers. Each layer of concrete is compacted by rodding 25 times with a standard steel rod. The top surface of the concrete is then leveled off with a steel rod. The mold is immediately pulled out from the concrete by raising it slowly and smoothly in a vertical direction, allowing the concrete to subside. The unsupported concrete cone will slump, and the difference between the height of the mold and the average height of the top surface of the slumped concrete is measured. The decrease in the height of the slumped concrete, measured to the nearest 5 mm, is called the slump.

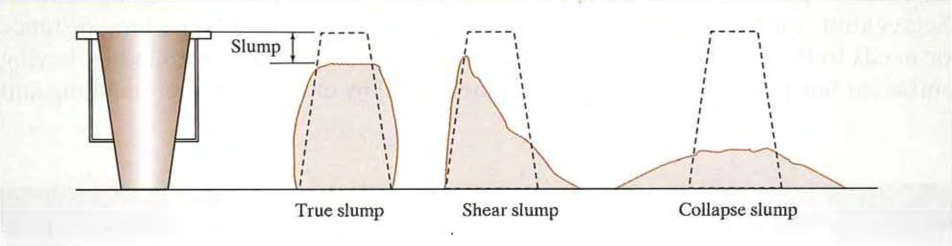
True slump is the one where concrete slumps uniformly without breaking, which indicates good cohesion in the mix (Figure 8.15). If one half of the concrete shears off in an inclined plane, it is called *shear slump*, and the test should be repeated again. If the repeated test again gives a shear slump, it indicates that the concrete mix is harsh and lacks cohesion. *Collapsed slump* indicates very high workability or very wet concrete. The typical range of slump for various construction types prescribed by ACI (American Concrete Institute) 211.1 (2009) is shown in Table 8.3.

Slump Loss

Slump loss is the decrease in concrete slump with time. Slump loss is caused by the stiffening of concrete due to evaporation of water, absorption of water by the aggregates, and the initial hydration of cement. Concrete with high initial slump is

True, shear, and collapse slump

Figure 8.15



(Courtesy of R. Tuladhar, James Cook University, Australia)

Recommended slumps for various types of construction

Table 8.3

Concrete construction	Slump (mm)	
	Maximum	Minimum
Reinforced foundation walls and footings	75 (3 in.)	25 (1 in.)
Plain footings, caissons, and substructure walls	75 (3 in.)	25 (1 in.)
Beams and reinforced walls	100 (4 in.)	25 (1 in.)
Building columns	100 (4 in.)	25 (1 in.)
Pavement and slabs	75 (3 in.)	25 (1 in.)
Mass concrete	75 (3 in.)	25 (1 in.)

(Based on ACI 211.1 (2009), "Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete," American Concrete Institute, Farmington Hills, Michigan, 38 pp.)

more likely to have greater slump loss. Slump loss is also influenced by the ambient temperature—the higher the ambient temperature, the greater the slump loss is. Slump loss is also higher if dry aggregates are used. Furthermore, use of supplementary cementitious materials can also affect the slump loss.

Slump loss should be monitored and controlled to ensure sufficient time for pouring and compaction of concrete. If the concrete needs to be transported to longer distances, mix with a higher slump is prepared. Slump loss is also controlled by reducing the temperature of the concrete. Alternately, retarding admixtures are also used to reduce slump loss.

8.4.3 Cohesiveness

The ease with which concrete mixture can be placed and finished without losing its homogeneity is termed cohesiveness. It is the resistance of the concrete against segregation and bleeding.

8.4.4 Early-Age Performance of Concrete

Segregation

Segregation is the separation of various ingredients of concrete mix resulting in a nonuniform distribution of the materials. Due to the difference in particle size and relative density, coarser aggregates tend to settle, and finer particles and water tend to rise upward (Figure 8.16). When the mixture is too dry, coarser aggregates tend to settle more than finer particles. The addition of water can help improve the cohesion of the mix and prevent segregation of this type. On the other hand, if the mix is too wet, cement paste tends to separate from the rest of the mixture, causing segregation. Segregation can be critical when concrete needs to be transported a long distance or needs to be pumped to a longer range. Segregation can be controlled by having sufficient fine particles (fine sand and cement) and by ensuring proper handling and

Figure 8.16 Segregation in precast panel

(Courtesy of M. Combe, Fibercon QLD, Australia) placing of concrete. Use of plasticizers and air-entraining agents can improve cohesion of concrete and reduce segregation.

Bleeding

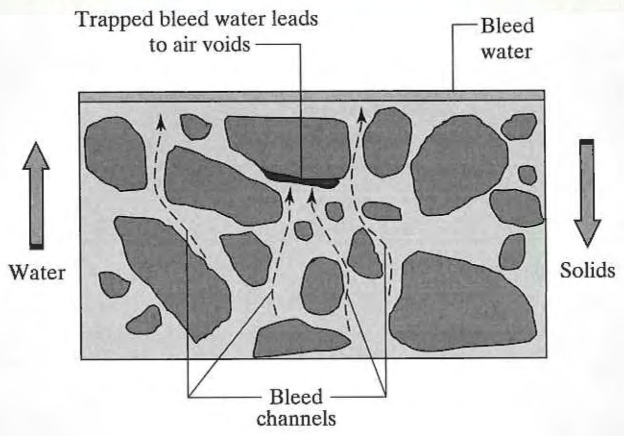
When fresh concrete is placed, the mix water has a tendency to rise upward (Figure 8.17). This upward movement of mix water separating from the rest of the concrete constituents is termed as bleeding. A limited amount of bleeding is beneficial to concrete. The upward movement of water removes the excess water present in the concrete, giving better strength to the concrete. Bleed water also ensures that the surface of the concrete does not dry and thus prevents plastic shrinkage. The plastic shrinkage effect is discussed in the next section. Bleed water on the surface also gives smooth finishing.

However, on the other hand, an excessive amount of bleed water can have a negative effect on the concrete. Bleed water can get trapped under reinforcing bars or aggregates, creating weak zones with air voids. Excessive bleeding can also cause plastic settlement cracking. Plastic settlement cracking occurs when the settlement of the aggregates is restrained (see the section "Plastic Settlement"). Furthermore, an undesirable amount of bleeding will also increase water/cement ratio on the top surface of the concrete, creating weak zones.

Bleeding can be controlled by increasing the cement fineness. Use of supplementary cementitious materials like fly ash or silica fume can also help to control bleeding. An increase in the rate of hydration will consume mix water at a faster rate and reduce the amount of bleeding. The rate of hydration reaction can be increased by using high-alkali content cement. Bleeding can also occur due to a high water/cement ratio. In such a case, reducing the water/cement ratio can help to control bleeding.

Bleeding in early-age concrete

Figure 8.17

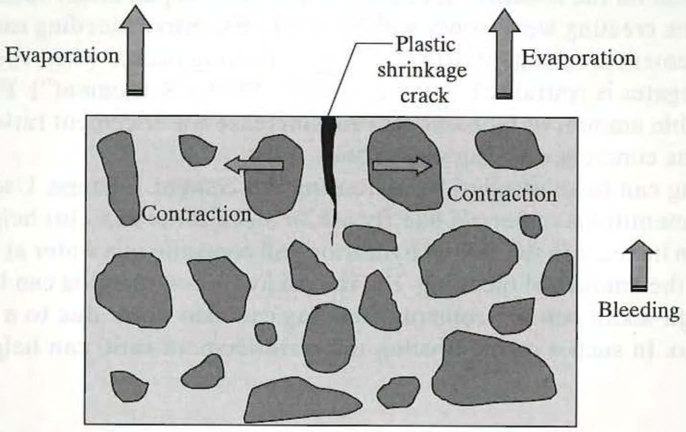


(Courtesy of R. Tuladhar, James Cook University, Australia)

Plastic Shrinkage

In a hot, dry, and windy environment, excessive drying can occur from the surface of the concrete. When evaporation of surface water exceeds bleeding, drying and contraction occurs especially on the surface (Figure 8.18). This phenomenon is known as plastic shrinkage, which can lead to the formation of surface cracking in random patterns or roughly diagonal/parallel patterns (Figure 8.19). These cracks, formed due to excessive loss of moisture from the surface, are called plastic shrinkage cracks. The amount of water loss from the surface of concrete is directly influenced by the ambient temperature, humidity, and wind, and temperature of the concrete. The rate of evaporation of concrete is generally restricted to 1 kg/m^2/hour (0.21 lb/ft^2/hour)

Figure 8.18 Plastic shrinkage and cracking



(Courtesy of R. Tuladhar, James Cook University, Australia)

Figure 8.19 Plastic shrinkage cracks on a concrete deck

(Courtesy of A. Vanderstaay, Department of Transport and Main Roads, Queensland Australia)

to avoid undesirable effects of plastic shrinkage. Plastic shrinkage cracking can be a critical issue while casting large, flat works like slabs in hot and dry weather condition. The large surface area of slabs can lead to faster evaporation of surface water.

Plastic shrinkage cracks start to appear within hours after placing concrete, when the concrete is still in plastic state. These cracks are typically 300 to 500 mm (12 to 20 in.) long and 3 to 5 mm (0.1 to 0.2 in.) wide, and are usually shallow. Plastic shrinkage cracks as such, may not cause structural problems; but excessive plastic shrinkage cracking can lead to chloride ion ingress into the concrete, which in turn will corrode the reinforcing steel embedded in the concrete.

Plastic shrinkage in concrete can be avoided by controlling the rate of evaporation from the surface of the concrete. Casting of concrete during extreme weather should be avoided, especially in a hot, dry, windy climate. Prewetting the formworks before concrete is poured will also cool down the surface and reduce excessive drying of the surface water. A concrete mix with a reasonable amount of bleeding will also help to reduce plastic shrinkage. Cooling of aggregates before mixing, using chilled water in the mix, and using aliphatic alcohols are some of the commonly used methods to avoid excessive plastic shrinkage. Micro-polypropylene fibers (Figure 8.20) are often mixed with concrete to reduce plastic shrinkage cracking.

Plastic Settlement

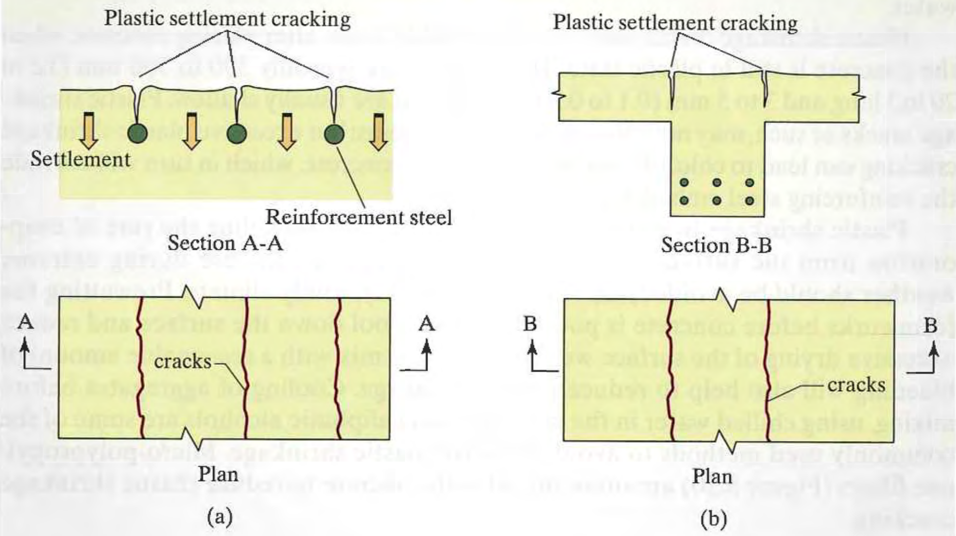
After pouring and compaction of fresh concrete, concrete tends to settle down. When this settlement is interrupted by the presence of reinforcement bars, large aggregates, or abrupt change in sections, plastic settlement cracks can occur (Figure 8.21). Settling of concrete is lower above reinforcement bars compared to either side of it.

Micro-polypropylene fiber

Figure 8.20

(Courtesy of R. Tuladhar, James Cook University, Australia)

Figure 8.21 Plastic settlement cracking due to restrain from (a) reinforcing steel (b) abrupt change in section



(Courtesy of R. Tuladhar, James Cook University, Australia)

This differential settlement causes concrete to crack directly over the reinforcements. Plastic settlement cracks can be avoided by reducing the water/cement ratio and ensuring proper compaction of the concrete.

8.5 Site Practice of Concrete

As soon as concrete is poured into the form, it needs to be properly compacted, finished, and cured to obtain the desired results.

8.5.1 Compaction

Freshly poured concrete needs to be compacted properly to remove air voids and to distribute concrete ingredients uniformly throughout the form. Compaction has to be carried out before the initial setting of the concrete. Concrete can be compacted by hand, using rodding or tamping or, more preferably, by using immersion vibrators (Figure 8.22), vibrating screeds, or external vibrators. Insufficient vibration can lead to air voids in the finished structure, which results in poor finishing and lower strength. However, excessive vibration can lead to segregation of concrete.

Inadequate compaction can lead to honeycombing in concrete structures (Figure 8.23). Honeycombing is exposure of coarse aggregate on the surface of the structure without sufficient mortar cover. Occurrence of honeycombing can be avoided by ensuring sufficient workability, by increasing fines in the concrete for proper packing, by avoiding reinforcement congestion, and by ensuring proper compaction.

Compaction of concrete using immersion vibrator

Figure 8.22

(Courtesy of R. Tuladhar, James Cook University, Australia)

Honeycombing in pier due to improper compaction of concrete

Figure 8.23

(Courtesy of A. Vanderstaay, Department of Transport and Main Roads, Queensland Australia)

8.5.2 Finishing

Finishing is done to create a concrete surface with the desired texture and smoothness. The type of concrete finishing depends upon the function of the structure. Warehouse concrete floors need to be very level, smooth, and durable. However, internal floors that will have floor coverings do not require such high-quality finishing.

Screeding and floating are first done to level the surface and remove extra concrete (Figure 8.24). Concrete is then left to set and bleed. Final finishing should be

Figure 8.24 Screeding and finishing of a concrete slab

(Courtesy of Dr. Shi Yin, James Cook University, Australia)

done only after bleeding is completed. Troweling is then done by hand or power trowel to produce a dense, hard, smooth, and durable surface. Edging will give stronger edges and prevent chipping. Joints are also placed at necessary locations during this stage.

8.5.3 Curing

After concrete is placed and finished, it needs to be cured to maintain sufficient moisture and temperature for proper hydration of cement. Proper curing is essential to obtain strong and durable concrete. Curing should be done immediately after finishing the concrete work. The duration of curing depends upon the desired properties of concrete, purpose of the structure, and ambient temperature. Generally, seven days of moist curing is recommended for field works.

Curing can be done by ponding water on the surface of the concrete. This method is usually used for flat works like slabs. Curing can also be done by spraying a fine mist of water on the surface of the finished concrete. Alternatively, the concrete surface is covered with wet burlap sacks or plastic sheets. It is important to ensure that concrete is evenly moist underneath the sheets. It is desirable to leave the form in place during curing to reduce water loss. Curing compounds are also often sprayed on the surface, which form a layer of impermeable membranes on the surface and reduce loss of water due to evaporation.

8.6 Properties of Hardened Concrete

With the proceeding of hydration reaction, concrete starts to gain strength and mechanical properties. Some of the main properties of hardened concrete that are of interest are strength, stiffness, durability, shrinkage, and creep. The properties of hardened concrete depend on the age of the concrete, the nature and proportion of the ingredients used in the mixture, the amount and distribution of pore space, and the quality of compaction and finishing.

8.6.1 Compressive Strength

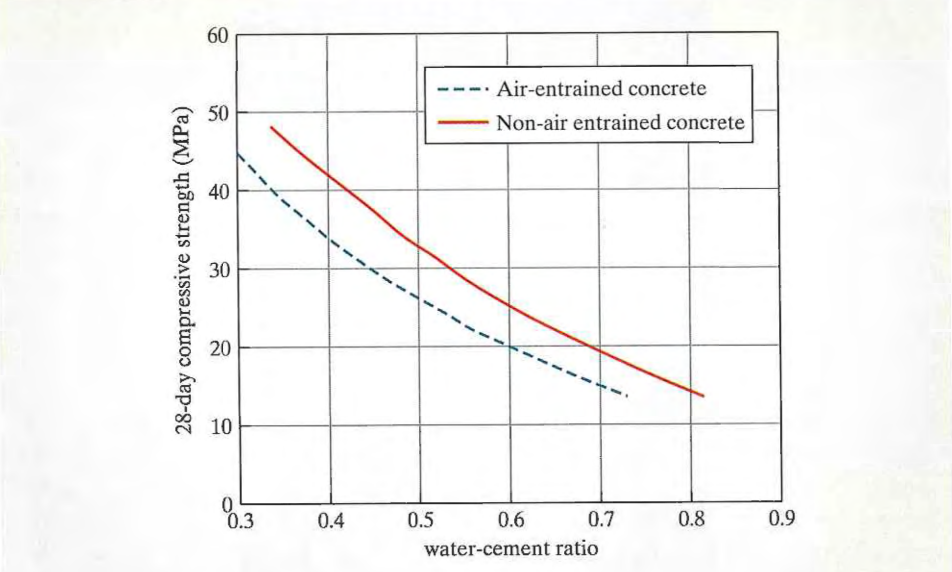
As discussed earlier, one of the main advantages of concrete is that it is very strong in compression. Due to its very high compressive strength, concrete is used widely as a construction material. Compressive strength is one of the most important properties of hardened concrete and is often used to define the quality of concrete. Compressive strength (f_c') is the ability of concrete to resist compressive stress and is measured in MPa (in SI units). For normal-strength concrete, the compressive strength may range from 20 to 60 MPa (2.9 to 8.7 ksi). High-strength concrete with compressive strength greater than 100 MPa (14.5 ksi) is often used for construction of large structures like high-rise buildings, and long-span bridges. High-strength concrete helps to reduce the member dimensions and to reduce the overall weight of the structure and cost of construction.

Hydration of cement and strength development in concrete continue for weeks, months, or even years. The strength of concrete increases rapidly in early age, and the rate decreases with the time. The strength at 28 days is considered as the standard strength of the concrete. By 28 days, concrete is considered to have developed most of its strength.

As discussed earlier, an increase in water/cement ratio adversely affects the strength of concrete (Figure 8.25). The higher the water-cement ratio, the lower the strength of the concrete is. An increase in cement, on other hand, can improve the strength of concrete. Using an optimum proportion of supplementary cementitious material like fly ash, silica fume, and ground granulated blast furnace also helps to increase the compressive strength of concrete. The fineness of the cement used in the concrete also affects the strength of concrete—use of finer cement particles increases the rate of hydration and results in a stronger concrete.

Variation of 28-day compressive strength with water-cement ratio for 19 to 25 mm (0.75 to 1.0 in.) nominal maximum size aggregates

Figure 8.25



(Based on Kosmatka, S.H., Kerkhoff, B., and Panarese, W.C. (2003), "Design and Control of Concrete Mixtures," Portland Cement Association, Skokie, Illinois, 340 pp.)

Compressive Strength Test

ASTM C39 specifies concrete cylinders of size 150 mm (6 in.) diameter and 300 mm (12 in.) long or 100 mm (4 in.) diameter and 200 mm (8 in.) long for compressive strength test. The sampling of concrete and casting of concrete cylinders are done according ASTM C192 (2007)/C31 (2010).

A cylinder mold is cleaned and the inside is coated lightly with oil. Concrete is poured into mold in three equal layers. Each layer of concrete is compacted by rodding 25 times with a steel rod. The top surface of concrete is then leveled off. Alternatively, cylinders may be compacted on a vibrating table. A clear identification tag is attached to the cylinder. The cylinders are left for setting at least for a day, and the molds are removed. The cylinders are moist cured until the time of testing. The top and bottom surface of the cylinders are often capped with sulphate mortar or high-strength gypsum plaster to give a smooth surface for testing.

Compressive strength tests are carried out on three or more specimens from each mix, at the prescribed age of concrete (usually 28 days). Additional compressive strength test at three and seven days are also often conducted to measure the early-age strength of concrete. The cylinders are subjected to uniaxial compression on a hydraulic testing machine, and the total force P required to crush the cylinders is measured, which divided by the area A of the cylinders gives the compressive strength of the concrete (Figure 8.26).

$$f_c' = \frac{P}{A} \tag{8.3}$$

where f_c' = compressive strength of concrete (MPa),

P = crushing load of the cylinder (N), and

Figure 8.26 Compressive strength test

(Courtesy of R. Tuladhar, James Cook University, Australia)

A = cross sectional area of the cylinder (mm^2).

Concrete cubes, instead of cylinders, are used for compression test in many European countries.

EXAMPLE 8.2

Three 150 × 300mm concrete cylinders from a construction site were tested at 28 days. The crushing loads for the cylinders are 630 kN, 659 kN, and 654kN. Calculate the average 28-day compressive strength of the concrete.

Use an empirical formula specified by ACI 318 (2011) to calculate modulus of elasticity of the concrete.

Solutions:

Specimen	Cross-sectional area (mm^2)	Crushing load (kN)	Compressive strength (MPa)	Average compressive strength (MPa)
1	17671.4	630	35.6	36.6
2		659	37.2	
3		654	37	

Hence, average compressive strength of the concrete $= f_c' = 36.6\ MPa$

Using the empirical equation specified in ACI 318 (2011) for modulus of elasticity,

$$E_c = 4731\sqrt{f_c'}\ MPa = 4731\sqrt{36.6} = 28,621\ \text{MPa}$$

8.6.2 Tensile Strength of Concrete

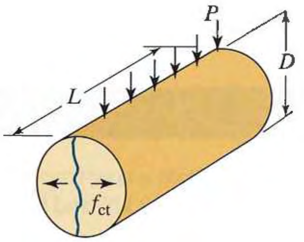
One of the main limitations of concrete is its low tensile strength. The tensile strength of concrete is almost one tenth of its compressive strength. The tensile strength of concrete is obtained from the indirect tensile tests as it is difficult to apply direct tensile force on the concrete specimens:

Split-Tension Test

A split-tension test is an indirect tension test method, which measures the splitting tensile strength (f_{ct}) of concrete. ASTM C496 (2011) specifies the use of a 150 mm (6 in.) diameter and 300 mm (12 in.) length cylinder for the test. The cylinder is laid horizontal on its side, and the load is applied from the top on the free side (Figure 8.27). The load that splits the cylinder into two halves is recorded. If P is the load at which the cylinder splits, L is the length of the cylinder, and D is the diameter of the cylinder; the splitting tensile strength of the concrete (f_{ct}) can be obtained as:

$$f_{ct} = \frac{2P}{\pi LD} \tag{8.4}$$

Figure 8.27 Split-tension test



(Courtesy of R. Tuladhar, James Cook University, Australia)

Flexural Strength Test

The *flexural strength* test measures the tensile strength of concrete in bending, and is often referred as the *modulus of rupture* (f_r). ASTM C78 (2010) specifies the flexural strength test on concrete beams of 150 mm (6 in.) by 150 mm (6 in.) cross-section and 500 mm (20 in.) length (Figure 8.28). The beam is subjected to a third point loading as shown in Figure 8.28 until it ruptures. The flexural tensile strength (f_r) is then calculated as:

$$f_r = \frac{M_{cr}}{Z} = \frac{PL}{bd^2} \tag{8.5}$$

where

f_r = flexural strength or modulus of rupture (MPa)

M_{cr} = maximum bending moment = $PL/6$

Z = section modulus of the beam = $bd^2/6$

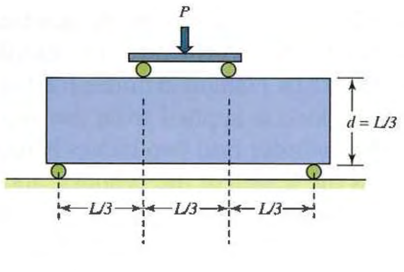
P = total load evenly distributed over the two loading points

L = beam span

b = width of the beam

d = depth of the beam

Figure 8.28 Flexural strength test by third-point loading



(Courtesy of R. Tuladhar, James Cook University, Australia)

The modulus of rupture is often calculated from the compressive strength (f_c') of concrete, using an empirical formula. ACI 318 (2011) recommends that for normal-weight concrete, the modulus of rupture (f_r) can be calculated as:

$$f_r = (0.62 \textit{ to } 0.83) \times \sqrt{f_c'} \tag{8.6}$$

where both f_r and f_c' are in MPa

EXAMPLE 8.3

Flexural strength test was conducted on the flexural beams of 150 × 150 × 500 mm size, from the same concrete mix as in Example 8.2, according to ASTM C78 (2010) under third-point loading. The span of the beam between the supports was 450 mm. The maximum load at which the beam failed was 35.3 kN. Calculate the flexural strength or modulus of rupture for the concrete. Compare it with the compressive strength calculated in Example 8.2.

Solutions:

Flexural strength for the beam can be calculated using Equation 8.5:

$$f_r = \frac{M_{cr}}{Z} = \frac{PL}{bd^2} = \frac{35.3 \times 1000 \times 450}{150 \times 150^2}\ MPa = 4.7 \text{ MPa}$$

Compared to the compressive strength of the concrete (Example 8.2), flexural strength is 7.8 times smaller than the compressive strength.

8.6.3 Modulus of Elasticity of Concrete

ACI 318 (2005) recommends that for normal-weight concrete the modulus of elasticity (E_c) can be found from compressive strength (f_c') using the following empirical equation:

$$E_c = 4731\sqrt{f_c'} \tag{8.7}$$

where both E_c and f_c' are in MPa.

8.6.4 Poisson's Ratio

Poisson's ratio (ν) for concrete is usually taken as 0.15 to 0.22. It is lower for high-strength concrete.

8.6.5 Creep in Concrete

Creep in concrete is another important phenomenon that affects the behavior of hardened concrete. Elastic deformation occurs in concrete structures as soon as a load acts on it. This causes elastic strains in concrete members. If the load is sustained for a long period of time, the strain in concrete gradually increases even without an increase in the load. This gradual increase in strain in concrete with time at a sustained loading is called as creep. Creep strain can be as high as three times the elastic strain. Creep does not usually affect the strength of a concrete structure, but it increases the deflection in concrete members and can cause serviceability issues under service load conditions.

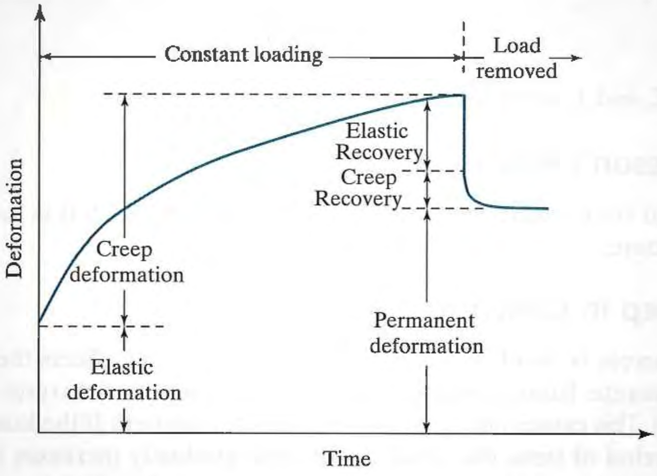
Figure 8.29 shows creep deformation in concrete subjected to compressive stress. As soon as the load is applied, elastic deformation occurs. At constant load, deformation in the concrete member increases with time due to creep in concrete. The creep deformation rate is higher at first, but the rate of increase in deformation decreases with time. If the load is removed, part of the elastic deformation is immediately recovered. The deformation is further recovered with time, which is called as creep recovery. However, some permanent deformation can remain in the structure even after the removal of the entire load.

Creep deformation increases with the duration of loading. Moreover, the higher the stress levels in the concrete members, the higher the creep is. Loading of concrete members at early age can also result in high creep deformations. An increase in aggregate content reduces the creep in concrete, whereas an increase in cement causes higher creep deformation. The types of aggregates used in the concrete also have a significant influence on the creep of concrete. Concrete with sandstone has higher creep than one with limestone.

8.6.6 Shrinkage in Concrete

The decrease in volume or the contraction of concrete due to loss of absorbed water is called shrinkage. Some of the water in the concrete mix does not undergo hydration reaction with cement and these water molecules are electrically bonded with concrete ingredients. Concrete can lose this absorbed water, causing loss in volume of the concrete. This is called as shrinkage or drying shrinkage. Drying shrinkage can be reduced by using a low water-cement ratio, maximum possible aggregate size, and shrinkage-limiting cement; and ensuring proper placing, compaction, and curing of concrete. If the concrete undergoing drying shrinkage is exposed to a wet environment again, it can partly regain the lost volume (Figure 8.30).

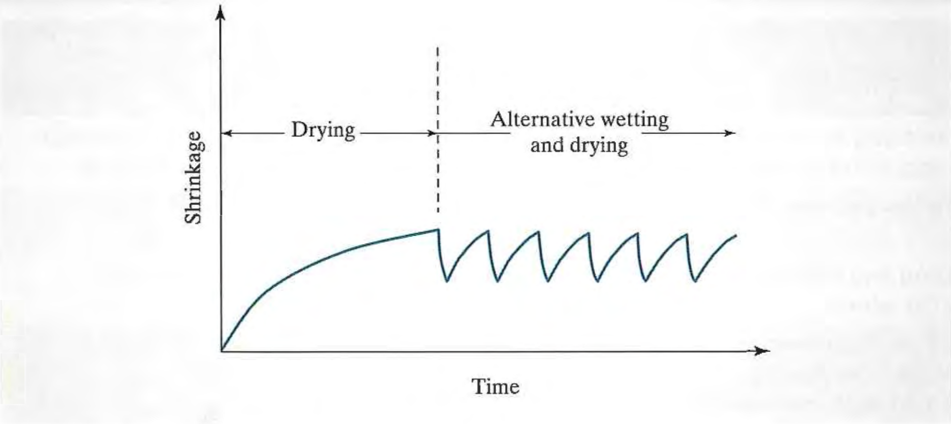
Figure 8.29 Creep deformation in concrete



(Courtesy of R. Tuladhar, James Cook University, Australia)

Shrinkage in concrete in drying environment

Figure 8.30



(Courtesy of R. Tuladhar, James Cook University, Australia)

If the contraction in a concrete member is restrained, tensile stresses are developed in the concrete. When this tensile stress exceeds the tensile strength of concrete, it cracks. In concrete slabs, drying shrinkage cracks can extend through the depth of the slab. These cracks can lead to water penetration and corrosion of reinforcements and compromise the durability of the structure. The building design codes often prescribe minimum necessary reinforcement in concrete slabs to control shrinkage cracking. Proper placement of contraction joints at appropriate intervals allows the contraction of concrete and, hence, can prevent shrinkage cracking.

8.6.7 Durability of Concrete

Durability of concrete can be defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration and its ability to maintain its desired engineering properties throughout its service life (ACI 201.2R, 2008). Along with strength, durability is an extremely important property of hardened concrete. Durability requirements can be the governing criteria for the design of concrete structures exposed to a severe environment. The ability of concrete to resist deterioration is influenced by the concrete ingredients used and the pouring and curing practices. Table 8.4 gives the maximum w/c ratio and minimum design strengths required for various exposure conditions as prescribed by ACI 318 (2011). Deterioration in concrete may be caused by chemical, physical, or mechanical attacks due to external or internal sources.

Carbonation

Carbonation occurs when carbon dioxide (CO_2) in the atmosphere penetrates the concrete and, in presence of moisture, reacts with hydroxides in cement (primarily calcium hydroxide), forming calcium carbonate ($CaCO_3$). The removal of $Ca(OH)_2$ results in significant reduction of alkalinity (pH) of the concrete, from about pH of 13 to as low as pH of 8.

$$\underset{\text{Byproduct of cement hydration}}{Ca(OH)_2} + \underset{\text{(from atmosphere)}}{CO_2} \xrightarrow{\text{Moisture}} \underset{\text{Calcium carbonate}}{CaCO_3} + H_2O$$

Table 8.4 Maximum water-cementitious material ratios and minimum design strengths for various exposure conditions (ACI 318, 2011)

Exposure condition	Maximum water-cementitious material ratio by mass for concrete	Minimum design compressive strength, f_c' (MPa)
Concrete not exposed to freezing and thawing, deicing chemicals, or any aggressive substances	Based on strength and workability requirements	Based on structural requirements
Concrete intended to have low permeability when exposed to water	0.50	28 (280)
Concrete exposed to freezing and thawing in a moist condition or to deicing agents	0.45	31 (310)
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, seawater, brackish water	0.40	35 (350)

Note: Values in parentheses are in ton/ft^2

(Based on ACI 318 (2011), "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, Farmington Hills, Michigan, 430 pp.)

The rate of carbonation is high when the atmospheric humidity is between 50 and 70 percent. At lower humidity (<25%), the degree of carbonation is considered insignificant, and at higher humidity (>75%), water inhibits CO_2 diffusion. Carbonation in good quality concrete, with sufficient reinforcement cover, does not have a detrimental effect on the structural properties of the concrete. However, reduction in the alkalinity of concrete can lead to corrosion of steel in the concrete. Generally, in reinforced concrete structures, a thin protecting layer of oxide film that forms around steel embedded in hydrating cement paste protects the steel from corrosion. If the pH of concrete falls below 11, this protective oxide film layer is broken, which can lead to corrosion of steel in presence of moisture and oxygen. ACI 318 (2011) provides guidelines on concrete cover required for different exposure conditions.

Chloride Attack

When the chloride content in concrete exceeds the threshold limit (about 0.15% water-soluble chloride by mass of cement), the protective oxide film layer around the steel is broken. This can lead to the corrosion of steel in the presence of moisture and oxygen. Corrosion of steel produces iron oxides, which cause expansion of steel in concrete. This induces internal stress and spalling of concrete (Figure 8.31).

Chlorides can be present in the concrete as contaminants of aggregates or mixing water. In coastal areas, concrete exposed to sea water can be subjected to chloride ions ingress. Use of certain accelerating admixtures, such as calcium chloride and deicing salts, can also introduce chloride into concrete.

The resistance of concrete to chloride ingress can be improved by lowering w/c ratio, proper curing (at least seven days of moist curing), and using SCMs, which reduces permeability of concrete. Providing sufficient concrete cover over the steel also helps protect against chloride ions reaching reinforcing steel.

Sulfate Attack

Sulfates and other salts in soil, ground water, or seawater can attack hardened concrete and have the potential to cause serious damage of concrete structures. Sulfates, such as calcium sulphate, sodium sulphate, and magnesium sulphate, can

Corrosion of reinforcement and concrete spalling

Figure 8.31

(Courtesy of R. Tuladhar, James Cook University, Australia)

react with aluminates, silicates, and calcium hydroxide in concrete, forming ettringite and gypsum. These expansive products can lead to expansion and cracking of concrete. Sulfate attack is more critical in concrete structures that are exposed to wetting and drying cycles than those exposed to continuously dry or wet conditions.

Sulfate attack in concrete can be mitigated by using low w/c ratio and using sulphate-resistant cements that have low C_3A content. Use of SCMs like fly ash is also an effective and economical method to improve sulphate resistance of concrete.

Alkali-Silica Reaction (ASR)

Aggregates containing reactive silica can react with alkali hydroxides from cement and other materials, causing expansion and deterioration of concrete. ASR can cause irregular superficial cracks with crack width ranging from 0.1 mm (0.004 in.) to 10 mm (0.4 in.) and less than 25 mm (1 in.) deep (Figure 8.32). ASR mostly affects the

Alkali-silica reaction (ASR)–induced cracking

Figure 8.32

(Courtesy of A. Vanderstaay, Department of Transport and Main Roads, Queensland Australia)

serviceability of structures and may not directly impact their strength. However, formation of ASR cracks can facilitate ingress of other harmful agents into the concrete, which can lead to deterioration of concrete and corrosion of steel in concrete.

ASR can be mitigated by using sufficient amount of SCMs. A very effective an economical method to prevent ASR is to partially replace portland cement with fly ash (>20%) in the concrete mix.

Freezing and Thawing Actions

Alternating freezing and thawing of concrete and the use of deicing salts can cause damage in concrete. Moisture present in concrete can freeze in very low temperatures, causing its expansion. An increase in hydraulic pressure due to expansion can cause deterioration of concrete.

Freezing and thawing damage can be controlled by using high-quality aggregates, a low w/c ratio, and air-entraining admixtures. Air-entraining admixtures create microscopic air bubbles in the paste. These tiny air-filled voids give space for expansion of water during the freezing and thawing cycles and help in relieving the hydraulic pressure generated. Air-entrained concrete (with 5% to 8% air content) and low w/c ratio is ideal for resisting the harmful effects of freezing and thawing actions. Table 8.8 displays the approximate air content percentage per cubic meter of concrete for non-air-entrained and air-entrained concrete.

8.7 Concrete Mix Design

As discussed in the preceding sections, the properties of fresh and hardened concrete are influenced by the type and proportions of the ingredients used in the concrete mix. It is, therefore, very important to carefully select constituent materials and calculate their proportions in the mix according to the specified requirements. Constituent materials should be chosen in such a way that the concrete mix is workable when fresh and strong and durable when hardened. It is also important that material prescribed for the mix should be readily available and economical as well. This process of selecting types and proportions of constituents of concrete to produce economical concrete with the desired workability, strength, and durability is called the mix design.

8.7.1 Mix Design Procedure

The basic steps for estimating mix proportion by absolute volume method presented by the Portland Cement Association (Kosmatka et al., 2003) are summarized here.

1. Calculate required average strength for the mix

Concrete for different structural uses will require different compressive strength. Hence, the first step of mix design is to identify the compressive strength of concrete, depending on the strength and durability requirement of the project. The specified (or design) compressive strength (f_c') for the project is shown in specifications and is used for all the structural design calculations. Unless otherwise specified, f_c' is based on 28-day tests on concrete cylinders. F_c' is specified for a particular project based on structural strength requirements. However, it should also meet the durability requirement of the structure. ACI 318 (2011) specifies that the design compressive strength (f_c') should be greater than the minimum design strength given in Table 8.4 for different exposure conditions.

The average strength of concrete used for construction should always be greater than the specified f'_c. The required average strength (f'_{cr}) for the concrete mix is therefore determined based on the probabilistic approach, taking into account the variability in materials, batching, and mixing of concrete; it is always greater than f'_c. This is to ensure that adequate concrete strength is developed in the structure.

As shown in Figure 8.33, ACI 318 (2005) specifies that 90 percent of the test results should have higher strength than the specified compressive strength (f'_c). Hence, for normal-strength concrete, the required average strength (f'_{cr}) for the mix design is taken as the larger of the values given by the following equations:

$$f'_{cr} = f'_c + 1.34s \tag{8.8}$$

$$f'_{cr} = f'_c + 2.33s - 3.45\,(for\ f'_c \le 35\ MPa)\ f'_{cr} = 0.90 \times f'_c + 2.33s\ (\text{for}\ f'_c > 35\ MPa) \tag{8.9}$$

where

f'_{cr} = specified compressive strength (MPa)

f'_c = required average compressive strength (MPa)

s = standard deviation (MPa)

The standard deviation (s) in Equations 8.8 and 8.9 should be determined from at least 30 sample strength tests. If 15 to 29 tests sample data are available, the modification factor for the standard deviation should be used as shown in Table 8.5.

If the number of samples taken is less than 15 or when data are not available to establish a standard deviation, Table 8.6 should be used to estimate required average compressive strength (f'_c).

2. Determine water-cement ratio

Once the required average compressive strength for the mix is known, the water-cement ratio is obtained from standard water/cement ratio versus strength charts specified in design standards as shown in Figure 8.25. For durability requirements,

Required average compressive strength obtained from probabilistic concept

Figure 8.33

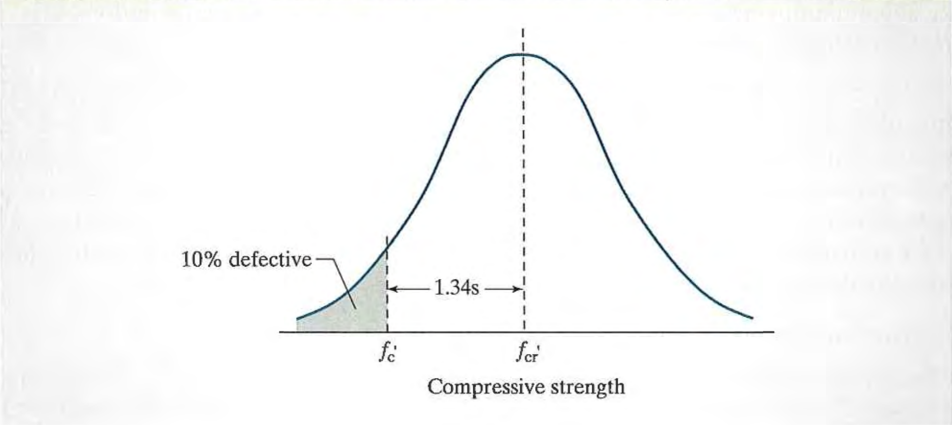


Table 8.5 Modification factor for standard deviation when less than 30 test samples are available

Number of test samples	Modification factor for standard deviation
15	1.16
20	1.08
25	1.03
30 or more	1.00

Note: Interpolate for the intermediate number of test samples

(Based on ACI 318 (2011), "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, Farmington Hills, Michigan, 430 pp.)

Table 8.6 Required average compressive strength (f'_{cr}) when the test samples are less than 15

Specified compressive strength (f'_c) in MPa	Required average compressive strength (f'_{cr}) in MPa
Less than 21	$f'_c + 7.0$
21 to 35	$f'_c + 8.5$
Over 35	$1.10 f'_c + 5.0$

(Based on ACI 318 (2011), "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, Farmington Hills, Michigan, 430 pp.)

the water-cement ratio should not exceed the maximum limits specified in ACI318 for different exposure conditions (Table 8.4).

3. Determine coarse aggregate size and grading

Large aggregates have lower surface area; hence, they require less cement paste to wet them. It is recommended to use largest maximum size aggregate from the workability perspective. The maximum size of aggregate is, however, limited by the dimension of the structure and spacing between the reinforcement bars. ACI 318 (2011) specifies that the nominal maximum size of aggregate should not be larger than:

a. 1/5 the narrowest dimension between sides of forms
b. 1/3 the depth of slabs
c. 3/4 minimum clear space between reinforcing bars or prestressing tendons
d. 1/3 of the slab thickness for unreinforced slabs

The fineness modulus of fine aggregate is chosen from coarse aggregate size and quantity of cement. Figure 8.7 shows the desirable grading zone for fine aggregate specified in ASTM C33 (2011a). From the fineness modulus of the fine aggregate and nominal maximum size of coarse aggregate, the volume of dry-rodded coarse aggregate per unit volume of concrete is determined using Table 8.7 as specified in ACI 211.1 (2009). The bulk volume of dry coarse aggregate multiplied by the bulk density of coarse aggregate gives the oven-dry mass of coarse aggregate.

4. Air content

The approximate air content percentage per cubic meter of concrete for non-air-entrained and air-entrained concrete for different nominal maximum aggregate size is obtained from Table 8.8 (ACI 318, 2005 and ACI 211.1, 2009).

Bulk volume of coarse aggregate per unit volume of concrete Table 8.7

Nominal maximum size of aggregate mm	Bulk volume of dry-rodded coarse aggregate per unit volume of concrete for different fineness moduli of fine aggregate			
	2.4	2.6	2.8	3.0
9.5 (0.37 in.)	0.5	0.48	0.46	0.44
12.5 (0.5 in.)	0.59	0.57	0.55	0.53
19 (0.75 in.)	0.66	0.64	0.62	0.60
25 (1 in.)	0.71	0.69	0.67	0.65
37.5 (1.5 in.)	0.75	0.73	0.71	0.69
50 (2 in.)	0.78	0.76	0.74	0.72
75 (3 in.)	0.82	0.80	0.78	0.76
150 (6 in.)	0.87	0.85	0.83	0.81

(Based on ACI 211.1 (2009), "Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete," American Concrete Institute, Farmington Hills, Michigan, 38 pp.)

Approximate air content percentage for non-air-entrained concrete for different nominal maximum sizes of aggregate Table 8.8

	Approximate air content percentage per cubic meter of concrete for different nominal maximum sizes of aggregate							
	9.5mm (0.37 in.)	12.5mm (0.5 in.)	19mm (0.75 in.)	25mm (1 in.)	37.5mm (1.5 in.)	50mm (2 in.)	75mm (3 in.)	150mm (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

(Based on ACI 211.1 (2009), "Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete," American Concrete Institute, Farmington Hills, Michigan, 38 pp.)

5. Determine slump

Slump indicates the consistency of concrete. According to the workability and consistency requirement, the desirable slump range is specified in the job specification. Alternatively, the approximate value of slump can be obtained from ACI 211.1 (2009) as shown in Table 8.3 according to the construction type.

6. Determine the water content

For the given slump, the nominal maximum size and shape of the aggregate and water content required in the mix is determined from Table 8.9 for non-air-entrained and air-entrained concrete.

7. Determine the cementing material content and type

After determining the water-cement ratio and water content, the total amount of cementitious material required is obtained by dividing the weight of water by the water-cement ratio.

Table 8.9 Approximate water content requirements for different slumps and nominal maximum sizes of aggregate

	Water (kg/m³) for indicated slump and nominal maximum aggregate sizes							
Slump (mm)	9.5 mm (0.37 in.)	12.5mm (0.5 in.)	19mm (0.75 in.)	25mm (1 in.)	37.5mm (1.5 in.)	50mm (2 in.)	75mm (3 in.)	150mm (6 in.)
Non-air-entrained concrete								
25–50 (0.2-2 in.)	207	199	190	179	166	154	130	113
75–100 (3-4 in.)	228	216	205	193	181	169	145	124
150–175 (6-7 in.)	243	228	216	202	190	178	160	-
Air-entrained concrete								
25–50 (1-2 in.)	181	175	168	160	150	142	122	107
75–100 (3-4 in.)	202	193	184	175	165	157	133	119
150–175 (6-7 in.)	216	205	197	184	174	166	154	-

Based on ACI 211.1 (2009), "Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete," American Concrete Institute, Farmington Hills, Michigan, 38 pp. and ACI 318 (2011), "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, Farmington Hills, Michigan, 430 pp.

8. Determine quantity of fine aggregate

Table 8.10 gives the estimate of the weight of one cubic meter of freshly mixed concrete for different nominal maximum aggregate size. The mass of water, cementitious material, and dry coarse aggregate required for one cubic meter of concrete is already known. The absolute volume of water, cement, and coarse aggregate can now be obtained by dividing the known mass of each constituent by their corresponding density.

In the absolute volume method, the volume of dry-fine aggregate is estimated by simply subtracting the absolute volume of all the other ingredients from the 1m³ concrete. The total mass of fine aggregate is determined by multiplying this volume by the density of fine aggregate.

Table 8.10 Density of freshly mixed concrete for different nominal maximum aggregate size

Nominal maximum aggregate size (mm)	Non-air-entrained concrete (kg/m³)	Air-entrained concrete (kg/m³)
9.5	2276 (142)	2187 (136)
12.5 (0.5 in.)	2305 (144)	2228 (139)
19.0 (0.75 in.)	2347 (146)	2276 (142)
25.0 (1 in.)	2376 (148)	2311 (144)
37.5 (1.5 in.)	2412 (151)	2347 (146)
50.0 (2 in.)	2441 (152)	2370 (148)
75.0 (3 in.)	2465 (156)	2394 (149)
150 (6 in.)	2507 (156)	2441 (152)

Note: Values in parentheses are in lb/ft³.

(Based on Mamlouk, M.S. and Zaniewski, J. P. (2011), "Materials for Civil and Construction Engineers," Prentice Hall, Upper Saddle River, New Jersey, p. 582.)

9. Moisture correction

Once the weight of all the ingredients is known, the final step is to correct moisture content and aggregate amount in the mix considering the existing moisture condition of the aggregate. Moisture conditions of coarse aggregate are shown in Figure 8.34. If dry aggregates are used in preparing concrete mix, they will absorb some part of the mix water. The water available for hydration will, therefore, decrease. On the other hand, if the aggregates used are wet or damp, they have excess moisture on the surface; the moisture is called free water. This free water is available for hydration reaction; hence, it will increase the water-cement ratio. The ideal moisture content of the aggregate is in a saturated surface dry (SSD) condition, where aggregates are saturated with moisture, but the surface is dry. Aggregates at SSD condition neither absorb water nor contribute water to the concrete mix.

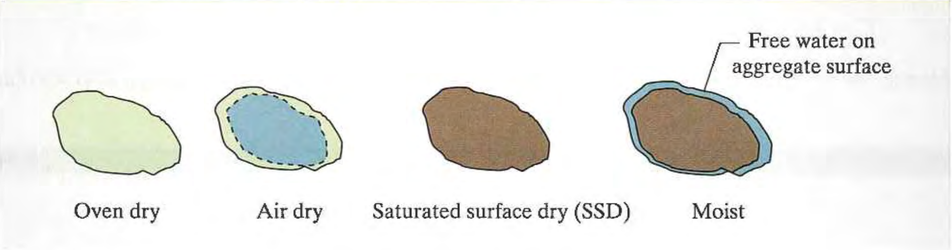
Aggregates used in concrete mix are seldom at SSD condition. If the aggregates used are dry, the amount of water that aggregates will absorb to reach SSD condition should be calculated and added on top of the water calculated for the mix. On the other hand, if the aggregates used are wet, the weight of free water on the surface of aggregates should be calculated and subtracted from the water calculated for the mix.

The moisture content at SSD condition and surface moisture of the aggregates used in the mix are determined according to ASTM C70 (2006) and C127 (2007). Typically, the moisture contents at SSD condition for coarse and fine aggregates are in the range of 0.2–4 percent and 0.2–2 percent, respectively. Free water contents for coarse and fine aggregates are generally in the range of 0.5–2 percent and 2–6 percent, respectively.

Finally, the weight of coarse and fine aggregate needs to be corrected as the previous calculations assumed to be in oven-dry condition.

Moisture content in aggregate

Figure 8.34



(Courtesy of R. Tuladhar, James Cook University, Australia)

EXAMPLE 8.4

The specified 28-day compressive strength of concrete for a project is $f'_c = 32$ MPa. The statistical data for the concrete production facility used for the project gives the standard deviation $(s) = 3.2$ MPa, obtained from more than 30 sample strength tests.

Calculate the required average compressive strength (f'_{cr}) for the concrete mix.

Using the compressive strength vs. water/cement ratio chart for the non-air-entrained concrete (Figure 8.18), calculate the water/cement ratio necessary for the mix.

Solutions:

The required average compressive strength is obtained as the larger value from Equations 8.8 and 8.9 according to ACI 318 (2011):

$$f'_{cr} = f'_c + 1.34s = 32 + 1.34 \times 3.2 = 36.3\ MPa\ (larger)$$
$$f'_{cr} = f'_c + 2.33s - 3.45 = 32 + 2.33 \times 3.2 - 3.45 = 36\ MPa$$

Hence, the required average compressive strength (f'_{cr}) of the concrete mix design is 36.3 MPa.

From Figure 8.18, for ($f'_{cr} = 36.3\ MPa$) and without air-entraining agents, the water-cement ratio needed is 0.38.

EXAMPLE 8.5

For the corrosion protection of reinforcement in a reinforced concrete structure exposed to seawater, the minimum specified compressive strength (f'_c) is 35 MPa. Determine the required average compressive strength (f'_{cr}) for the concrete mix design. The standard deviation (s), computed from 20 test samples, is 3.9 MPa.

Solution:

The specified compressive strength of concrete from durability requirement (f'_c) = 35 MPa.

The standard deviation (s) obtained from 20 test samples = 3.9 MPa.

Since the standard deviation is computed from 20 test samples only, the modification factor for standard deviation = 1.08 (Table 8.6).

The required average compressive strength is obtained as the larger value from Equation 8.8 and 8.9 according to ACI 318 (2011):

$$f'_{cr} = f'_c + 1.34s = 35 + 1.34 \times (3.9 \times 1.08) = 40.6\ MPa$$
$$f'_{cr} = f'_c + 2.33s - 3.45 = 35 + 2.33 \times (3.9 \times 1.08) - 3.45 = 41.4\ MPa\ (larger)$$

Hence, the required average compressive strength (f'_{cr}) of the concrete mix design is 41.4 MPa.

EXAMPLE 8.6

A design compressive strength (f'_c) of 30 MPa is specified for construction of internal reinforced concrete beams not exposed to any harsh environment. Following are the specifications for the project:

a. Cement: Ordinary portland cement with relative density 3.1.

b. Fine aggregate: Natural sand with oven-dry relative density of 2.45; SSD moisture content of 0.5%; moisture content in the supplied sample is 5.7%; fineness modulus 2.4.

c. Coarse aggregate: 25 mm nominal maximum size rounded gravel; oven-dry relative density of 2.53; SSD moisture content of 1.2%; oven-dry rodded bulk density of 1800 kg/m³, moisture content in the supplied sample is 3.2%.

d. No admixtures to be used.

e. Slump should be between 50–100 mm.

f. Statistical data from the concrete plant from sample tests on more than 30 samples shows standard deviation (s) of 4.8 MPa.

Perform mix design to obtain proportions of cement, water, and fine and coarse aggregate for one cubic meter of concrete.

Solution:

a. Strength:

As the structure is not exposed to any harsh environment, design compressive strength is governed by the strength requirements. Thus,

Design compressive strength (f_c') = 30 Mpa.

The required average compressive strength (f_{cr}') is obtained as the larger value from Equation 8.8 and 8.9 according to ACI 318 (2005):

$$f_{cr}' = f_c' + 1.34s = 30 + 1.34 \times 4.8 = 36.4 \text{ MPa}$$
$$f_{cr}' = f_c' + 2.33s - 3.45 = 30 + 2.33 \times 4.8 - 3.45 = 37.7 \text{ MPa } (\textit{larger})$$

Hence, the required average compressive strength (f_{cr}') of the concrete mix design is 38 Mpa.

b. Water-cement ratio:

From Figure 8.18, for f_{cr}' = 38 MPa and without air-entraining agents, the water-cement ratio needed is 0.45.

c. Air content:

For non-air-entrained concrete with nominal maximum aggregate size of 25 mm, the approximate percentage of air content per cubic meter of concrete is taken from Table 8.6 as 1.5 percent.

d. Water content:

For 50–100 mm slump and 25 mm nominal maximum size aggregate, the water content required in the mix is determined from Table 8.7.

$$\text{Water content} = 193 \text{ kg/m}^3$$

e. Cement content:

Cement content is water content/(w/c ratio) = 193/0.45 = 428 kg/m^3.

f. Coarse aggregate content:

For 25 mm nominal maximum size coarse aggregate and fineness modulus of 2.4, the bulk volume of dry-rodded coarse aggregate per unit volume of concrete is determined from Table 8.4 as 0.71.

The oven-dry rodded bulk density for aggregate is given as 1800 kg/m^3.

Hence, the oven-dry mass of coarse aggregate for 1m^3 of concrete

$= 1800 \times 0.71 = 1278$ kg

g. Fine aggregate content:

The volume of fine aggregate can be determined by subtracting the absolute volume of all other ingredients from the 1m^3 concrete.

Volume of each ingredient already known:

$$\text{Water} = 193/1000 = 0.93 \text{ m}^3$$
$$\text{Cement} = 428/(3.1 \times 1000) = 0.138 \text{ m}^3$$
$$\text{Air} = 1.5/100 = 0.015 \text{ m}^3$$
$$\text{Coarse aggregate} = 1278/(2.53 \times 1000) = 0.505 \text{ m}^3$$

Total volume of known ingredients = 0.851 m^3

Hence, the absolute volume of fine aggregate = $1 - 0.851 = 0.149$ m^3
Thus, the mass of dry fine aggregate = $0.149 \times 2.45 \times 1000 = 365$ kg

h. The proportions of the different ingredients are:

Water = 193 kg

Cement = 428 kg

Coarse aggregate (dry) = 1278 kg

Fine aggregate (dry) = 365 kg

Total mass = 2264 kg

i. Moisture correction:

With the moisture content of 5.7 percent and 3.2 percent in the supplied fine and coarse aggregates, respectively, the weights of aggregates to be used in batching are:

$$\text{Fine aggregate } (5.7\%) = 365 \times 1.057 = 386 \text{ kg}$$
$$\text{Coarse aggregate } (3.2\%) = 1278 \times 1.032 = 1319 \text{ kg}$$

Free water contributed by the aggregates is

$$\text{Fine aggregate} = 5.7\% - 0.5\% = 5.2\%$$
$$\text{Coarse aggregate} = 3.2\% - 1.2\% = 2\%$$

Hence, taking free water into consideration, total water required to be added is

$$= 193 - (365 \times 0.052) - (1278 \times 0.02) = 148 \text{ kg}$$

After moisture correction, final estimated batch weights for one cubic meter of concrete:

Water: 148 kg
Cement: 428 kg
Coarse aggregate (3.2% moisture content): 1319 kg
Fine aggregate (5.7% moisture content): 386 kg
Total weight: 2281 kg

8.8 Summary

1. Concrete is the most widely used construction material. The chief constituent materials of concrete are portland cement, water, and coarse and fine aggregates. Apart from these basic ingredients, admixtures can be added to a concrete mix to alter its specific properties.
2. Portland cement is manufactured by burning lime and clay in kilns at a very high temperature.

3. When portland cement is mixed with water, the compounds of cement chemically react with water to form calcium silicate hydrates that are chiefly responsible for gain in strength in cement paste.
4. Supplementary cementitious materials like fly ash, silica fume, ground granulated blast furnace slag, or natural pozzolan can also be used in addition to or in partial replacement of portland cement in concrete.
5. Important properties of fresh concrete are workability, consistency, and cohesiveness. After placing and finishing concrete work, it is important to properly cure it to ensure sufficient moisture and temperature condition for proper hydration of cement to occur.
6. Concrete has to be transported, placed, compacted, and finished when it is still in the plastic state.
7. With the progression of hydration reaction, concrete develops mechanical strength and hardens. Strength and durability are the two main properties of hardened concrete.
8. The process of selecting the types and proportion of constituent materials of concrete to produce economical concrete with the desired workability, strength, and durability is called the mix design. With a proper mix design, proper placing, compaction, and curing, a very strong and durable concrete can be achieved.

Exercises

1. Which one of the following four main compounds in cement is responsible for the long-term strength gain in concrete?
   - **a.** Tricalcium silicate (C_3S)
   - **b.** Dicalcium silicate (C_2S)
   - **c.** Tricalcium aluminate (C_3A)
   - **d.** Tetracalcium alumino-ferrite (C_4AF)
2. The loss of excessive moisture from the surface of concrete in plastic state leads to:
   - **a.** Segregation
   - **b.** Bleeding
   - **c.** Plastic shrinkage
   - **d.** Plastic settlement
3. The chemical reaction between water and cement that results in rapid strength development is known as:
   - **a.** Hydration
   - **b.** Segregation
   - **c.** Bleeding
   - **d.** Shrinkage
4. Which of the following types of water is not suitable for using in the concrete mix for reinforced concrete structures?
   - **a.** Tap water
   - **b.** Sea water
   - **c.** Recycled concrete wash water
   - **d.** All of the above

5. Materials that react chemically with calcium hydroxide, a byproduct of hydration reaction, in the presence of moisture to form cementitious compound are called:
   a. Hydraulic materials
   b. Pozzolanic materials
   c. Portland cement
   d. Calcium silicate hydrate
6. The main source of fly ash is:
   a. Cement industry
   b. Nuclear power plants
   c. Thermal power plants burning coal
   d. Iron industry
7. During the cement production process, 3 to 5 percent gypsum is added to:
   a. Accelerate setting time of cement
   b. Increase strength of cement
   c. Achieve the typical gray color of cement
   d. Retard setting time of cement
8. Micropolypropylene fiber is used in concrete to:
   a. Reduce water/cement ratio
   b. Increase tensile strength of concrete
   c. Prevent plastic shrinkage cracking
   d. Prevent corrosion of reinforcement
9. An alkali-silica reaction (ASR) in concrete can be prevented by:
   a. Using accelerators like calcium chloride
   b. Partially replacing portland cement with more than 20 percent fly ash
   c. Using aggregates with a higher percentage of silica
   d. Adding air-entraining admixtures
10. Production of 1000 kg of cement produces approximately:
    a. 1 kg of CO_2
    b. 10 kg of CO_2
    c. 100 kg of CO_2
    d. 1000 kg of CO_2
11. What are supplementary cementitious materials? What are the benefits of using supplementary cementitious material in concrete?
12. Contact local concrete batching plants in your area and inquire about the types of supplementary cementitious materials they regularly use in their concrete mixes. Write a report on the effects of these supplementary cementitious materials in the properties of fresh and hardened concrete.
13. Why is a moderate level of bleeding beneficial for concrete?
14. What are different types of admixtures used in concrete? Define their roles in altering various properties of concrete.
15. What is the difference between plastic shrinkage and drying shrinkage in concrete? What different measures can be taken to minimize plastic shrinkage cracking in concrete?

16. Define workability. What are the different factors that affect the workability of concrete mix?

17. What are the different challenges of placing concrete in hot weather?

18. Listed below are specialized types of concrete that are used for specific constructions. These topics have not been covered in this chapter. Search for the relevant literature on these specialized concrete types and write a two-to-three page report on them:

a. Self-compacting concrete
b. Fiber-reinforced concrete
c. Photocatalytic concrete
d. High-strength concrete

19. As a concrete plant engineer, you are asked to design a concrete mix to withstand freezing and thawing environmental conditions. What measures will you take to ensure adequate freeze/thaw resistance?

20. A split-tension test was conducted on a concrete cylinder with a 150 mm diameter and 300 mm length. The load that split the cylinder is 268.5 kN. Calculate the splitting tensile strength (f_{ct}) of the concrete.

21. The third-point flexural strength test was conducted on a flexural beam of 150 mm by 150 mm cross section and 450 mm span. The maximum load at failure was 29.2 kN. Calculate the flexural strength of the concrete.

22. The crushing loads for three concrete cylinders in compression tests carried out at 28 days are 212 kN, 238 kN and 230 kN. All the cylinders are of 100 mm diameter and 200 mm length. Calculate the average 28-day compressive strength and modulus of elasticity of the concrete.

23. To determine the moisture content, samples of fine and coarse aggregate from the supplied source were collected in measuring trays and were dried in an oven. The initial weights and oven-dry weights of sample aggregates in the tray are shown in Table 8.11. Determine the moisture content in the given fine and coarse aggregates.

Assume that the saturated surface dry (SSD) moisture conditions for fine and coarse aggregate are 1 percent and 2 percent, respectively. Determine the total free-water (extra moisture) in the concrete mix, if 220 kg of fine aggregate and 425 kg of coarse aggregate from the given source were to be used for preparing concrete mix.

Classification of coarse aggregate — Table 8.11

Material	Weight of wet sample + tray (gm)	Weight of dry sample + tray (gm)	Weight of the tray (gm)
Fine aggregate	1040	1005	208
Coarse aggregate	1533	1516	206

24. Table 8.12 represents data from sieve analysis carried out on a coarse aggregate sample. Determine the fineness modulus and plot the grading curve for the given coarse aggregate.

Table 8.12 Sieve analysis of concrete coarse aggregate

ASTM Sieve No.	Sieve size	Mass retained (g)
3/4 in	19mm	4640
3/8 in	9.5mm	4950
4	4.75mm	3790
8	2.36mm	260
16	1.18mm	120
30	600µm	0
50	300µm	0
100	150µm	0
Pan		0

25. As a plant engineer, you are required to determine the required average compressive strength for a construction project where the specified compressive strength from structural requirement is 30 MPa. Sufficient data is not available for computing standard deviation. Based on the required average compressive strength, determine the w/c ratio for the mix using Figure 8.22.

26. Determine the different proportions of constituents required for one cubic meter of concrete using the absolute volume method. From structural considerations, a design compressive strength (f_c') of 28Mpa is required for construction of concrete structures. Following specifications are also prescribed in the design details:

- **a.** Cement: Ordinary portland cement with relative density of 3.15
- **b.** Fine aggregate: Natural sand with oven-dry relative density of 2.3; SSD moisture content of 0.2 percent; moisture content in the supplied sample 4.3 percent; fineness modulus 2.63
- **c.** Coarse aggregate: 19 mm nominal maximum size crushed stone; oven-dry relative density of 2.8; SSD moisture content of 1.8 percent; oven-dry rodded bulk density of 1950 kg/m3, moisture content in the supplied sample is 4.9 percent.
- **d.** Slump should be between 25–75 mm.
- **e.** Use standard deviation (s) as 4.8 Mpa for calculating required average compressive strength.

27. The following requirements are specified for concrete for construction of a reinforced concrete jetty exposed to sea water and a moderate freeze-thaw environment.

Characteristic compressive strength at 28 days	35 Mpa
Slump required	75 to 100mm
Maximum coarse aggregate (crushed stone) size	25mm
Oven dry relative density of coarse aggregate	2050 kg/m^3

Moisture content in coarse aggregate at SSD	0.9%
Moisture content of the coarse aggregate stock	2%
Oven-dry relative density of fine aggregate	2.52
Moisture content of the fine aggregate at SSD	0.7%
Moisture content of the fine aggregate stock	5.2%
Fineness modulus	2.5

The recommended dosage of air-entraining admixture is 0.4g per kg of cement.

Standard deviation (*s*) for the concrete strength for the plant is unknown, as no statistical data on previous concrete mixes from the plant are available. Determine the different proportions of constituents required for one cubic meter of concrete, using the absolute volume method.

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Paris's magnificent Eiffel Tower is composed of wrought iron, an iron alloy with low carbon. Courtesy of M. Bobby Kannan, James Cook University, Australia

CHAPTER

9

Metals and Alloys

9.1 Introduction

Metals and alloys are the backbone of engineering structures. They are broadly classified into ferrous (iron-based) and nonferrous materials. Materials such as steels, stainless steels, aluminum-based, magnesium-based, and copper-based alloys have been used in structural and chemical industries for many decades. The concept of alloying, that is, combining two or more metals or addition of small amount of nonmetallic elements such as carbon, nitrogen, and oxygen to a base metal, has led to the development of high performance engineering alloys. Heat treatment is a versatile technique that allows tailoring the properties of metals and alloys for specific applications. In fact, alloy composition has a significant influence on the property of materials undergoing heat treatment. In general, metals and alloy components under extreme conditions such as high temperatures, high tensile/compressive loads or exposure to corrosive environments can undergo failure. In this chapter, the various types of ferrous and nonferrous alloys, heat-treatment procedures, and also the failure mechanisms such as fracture, fatigue, creep, and corrosion of these materials are covered.

9.2 Ferrous Alloys

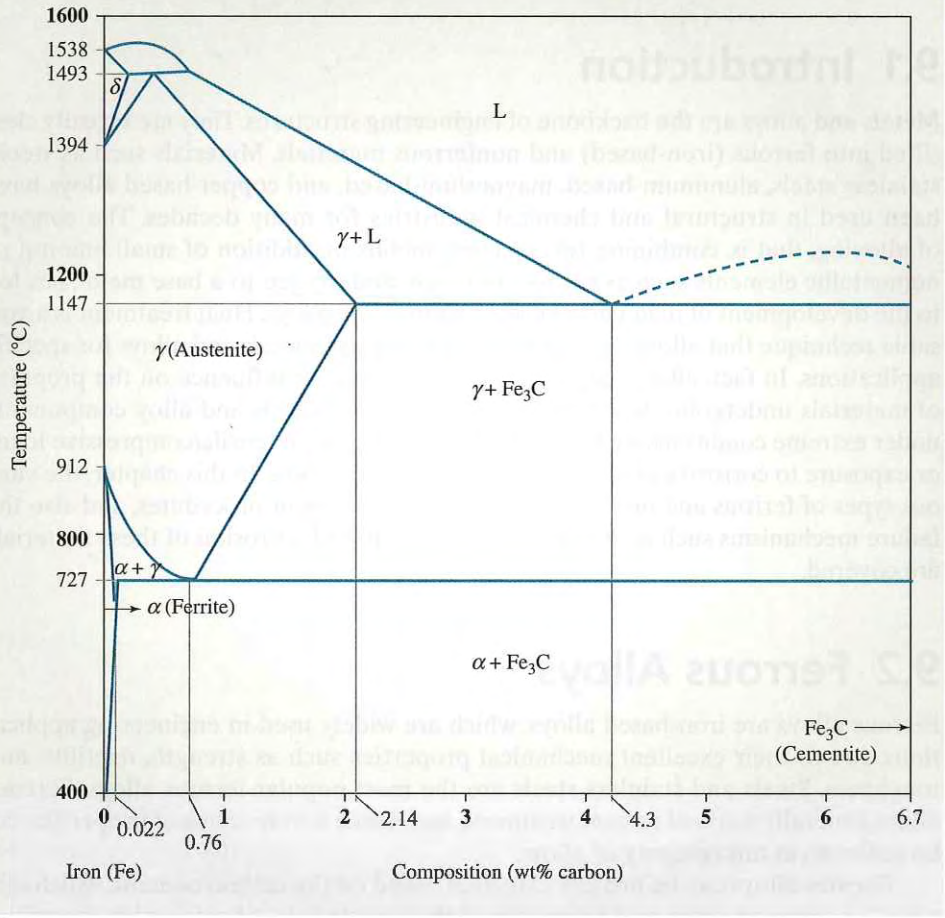
Ferrous alloys are iron-based alloys, which are widely used in engineering applications due to their excellent mechanical properties such as strength, ductility, and toughness. Steels and stainless steels are the most popular ferrous alloys. Ferrous alloys generally respond to heat treatment, and hence a wide-range of properties can be achieved in this category of alloys.

Ferrous alloys can be broadly classified based on the carbon content, which dictates the microstructure and properties of these materials. Alloying with chromium has led to a popular class of materials known as stainless steels. Due to their high corrosion resistance, stainless steels have found application not only in engineering structures/components but also in domestic appliances.

9.2.1 Iron-Carbon Diagram

An iron-carbon diagram is a phase diagram of iron-carbon alloy which illustrates the phases present at various temperatures (Figure 9.1). In pure iron, α-iron, also known as the ferrite phase, is stable below 912 °C. Ferrite has a body-centered cubic (BCC) crystal structure, and contains a small amount of carbon with a maximum solubility of about 0.022 percent by weight (wt%) at 727 °C. The carbon atoms are situated interstitially, and since the size of a carbon atom is larger than the interstitial gaps, a local strain field is present around the carbon atoms, which consequently increases the strength of the alloy. Mild steels contain up to about 0.2 wt% carbon, which is mainly ferrite phase. As the carbon content of the steel is increased, the amount of pearlite phase, which is a fine lamellar structure of ferrite and cementite (Fe_3C), also increases. Austenite (γ-iron) is a solid solution of carbon in iron and has a face-centered cubic

Figure 9.1 Iron-carbon phase diagram



(Adapted from Callister W. D., and Rethwisch D.G. 2010)

(FCC) crystal structure. This phase forms at relatively high temperatures. Basically, there are three reactions in the iron-carbon diagram:

1. *Peritectic Reaction:* The alloy with 0.15 wt% carbon at 1493 °C consists of solid δ-iron and liquid phase, and upon cooling will transform to γ-iron.
2. *Eutectic Reaction:* The alloy with 4.3 wt% carbon at 1147 °C is in a liquid phase, and upon cooling will transform into γ-iron and Fe_3C.
3. *Eutectoid Reaction:* The alloy with 0.76 wt% carbon at 727 °C is γ-iron, and upon cooling will transform to α-iron and Fe_3C.

An alloy (solute and base metal) which results in a homogeneous single microstructure is called eutectoid. If an alloy constitutes two or more different microstructures, then depending on the solute concentration, the mixture is called hypoeutectoid (less solute) or hypereutectoid (more solute).

Eutectoid steel: This steel contains 0.76 wt% carbon, and when it is cooled from a single phase austenite, the steel forms two phases: ferrite and cementite. The resulting microstructure is layered and is called pearlite.

Hypoeutectoid steel: It contains less than 0.76 wt% carbon. When the steel is cooled from austentite transformation temperature, proeutectoid-ferrite forms and grows until eutectoid composition is reached, and the remaining microstructure forms pearlite.

Hypereutectoid steel: This steel contains more than 0.76 wt% carbon. When it is cooled, cementite formation begins, and the remaining steel when reaching eutectoid composition forms pearlite. Hypereutectoid steel is much harder than hypoeutectoid steel, since cementite is harder than ferrite.

9.2.2 Steels

Based on the carbon content and alloying elements, steels are classified as (1) low carbon steel, (2) mild steel, (3) medium carbon steel, and (4) high-strength steel. Table 9.1 shows the designations for steels by the American Iron and Steel Institute (AISI). The composition of the steel is denoted by a four-digit number, where the first two digits denote the alloying elements and the last two digits the carbon content. For example, 1025 steel stands for 0.25 wt% carbon-containing plain carbon steel.

Table 9.1 AISI classification of steels

AISI Classification	Type of Steel
10xx	Plain carbon steels
11xx	Resulphurized steels
13xx	Manganese steels
40xx	Molybdenum steels
41xx	Chromium-Molybdenum steels
43xx	Nickel-Chromium-Molybdenum steels
51xx	Chromium steels
61xx	Chromium-Vandium steels
92xx	Silicon-Manganese steels

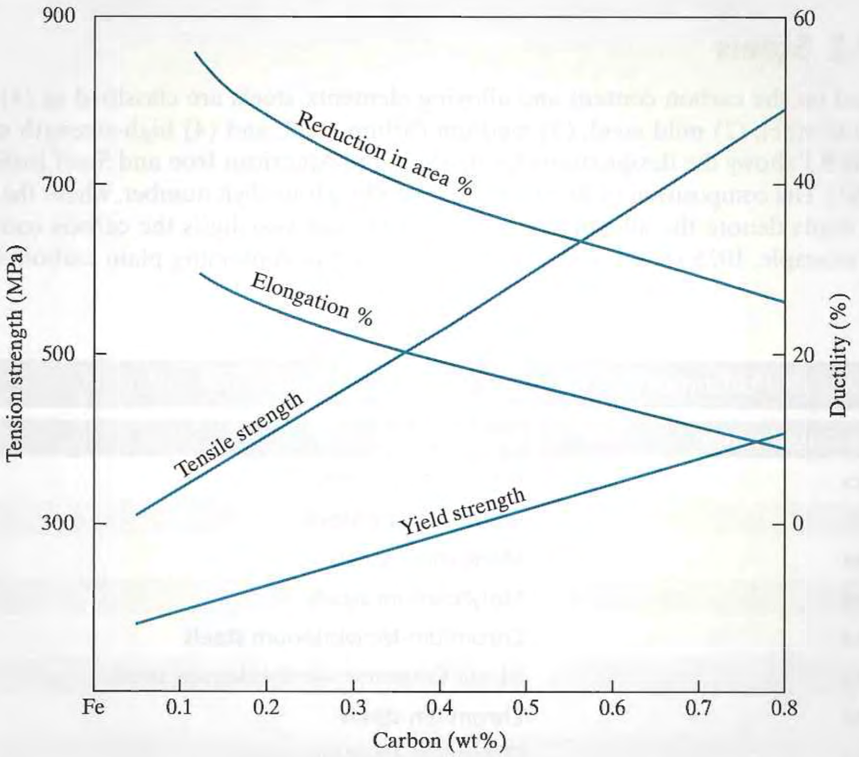
The carbon content of the steel strongly influences the mechanical properties such as yield strength, tensile strength, and ductility. As shown in Figure 9.2, as the yield strength and tensile strength increase with an increase in carbon content, the ductility, that is, the percentage of elongation and reduction in the area (RA), decreases.

Low carbon steels contain carbon content of about 0.1 wt%, and they are not heat treatable. Their typical mechanical properties are: yield strength (200–300 MPa or 2000–3000 ton/ft^2), ultimate tensile strength (300–370 MPa or 3000–3700 ton/ft^2), and elongation (28–40%). The microstructure of these steels mainly contains ferrite and a small amount of pearlite. These steels are used in corrugated sheets, automobiles, and tin cans for their excellent formability.

Mild steels contain carbon content of 0.15–0.25 wt%, which are also not heat treatable. Due to the relatively high carbon content, the strengths of these steels are higher than those of low carbon steels. Their typical mechanical properties are: yield strength (300–350 MPa), ultimate tensile strength (400–450 MPa or 4000–4500 ton/ft^2), and elongation (26–30%). Mild steels have excellent weldability. These steels are widely used in beams (I and H), angles, channels, oil pipelines, and boilers.

To improve the strength of mild steels, a small amount of special elements such as niobium, vanadium, or titanium is added, which forms strong carbides. These steels are called microalloyed steels. They contain very fine ferrite grains which increase

Figure 9.2 Mechanical properties of steels as a function of carbon content



(Adapted from Raghavan V., 1997)

the strength. Further, the fine distribution of carbides causes *precipitation hardening*. Copper and phosphorus are added to mild steels to improve the atmospheric corrosion resistance. These elements facilitate the formation of a crack-free FeOOH layer that prevents oxygen and water contact with the base iron metal. These steels are known as *weathering steels*. Sulphur and manganese are added to mild steels to improve their machinability. These elements form globular MnS particles, which break the continuity of the matrix and promote chip formation. These steels are called resulphurized grade steels or *free cutting steels*.

Medium carbon steels have carbon content in the range of 0.3–0.65 wt%. These steels are used in railway rails and wheels, typically after mechanical or thermo-mechanical treatment such as forging or hot rolling. They contain mainly pearlitic microstructure, which results in a hard, strong, and good wear-resistant steel. When medium carbon steels (0.5–0.65 wt%) are heat-treated, that is, quenched (rapid cooling) and tempered (controlled heating), they achieve high yield strength. These steels are known as spring steels. Silicon and vanadium are added to mild carbon steels, which provide stress relief during tempering and prevent grain coarsening during austenizing treatment, respectively.

High-strength steels. Steels which have tensile strength over 600 MPa (6000 ton/ft^2) are called high-strength steels. High-strength low-alloy (HSLA) steels contain small amounts of alloying elements and a carbon content of 0.15–0.25 wt%. Their typical mechanical properties are: yield strength (400–700 MPa or 4000–7000 ton/ft^2), ultimate tensile strength (500–800 MPa or 5000–8000 ton/ft^2), and elongation (18–25%). The microstructure of these steels contains ferrite and bainite or tempered martensite and bainite. Steels with greater than 1000 MPa (10,000 ton/ft^2) tensile strength are called ultra-high strength steels. Nickel-chromium-molybdenum (Ni-Cr-Mo) low-alloy steel is an ultra-high-strength steel, where nickel increases the toughness of ferrite; chromium provides hardness, strength, and wear resistance; and molybdenum is added to counter temper brittleness. When this steel (AISI 4340) is quenched and tempered, it exhibits the following mechanical properties: yield strength (1200 MPa or 12,000 ton/ft^2), ultimate tensile strength (1350 MPa or 13,500 ton/ft^2), and elongation (15%). Maraging steels are another group of ultra-high-strength steels. These steels contain 18 wt% nickel, 2–3 wt% cobalt, 2–3 wt% molybdenum, 0.03 wt% carbon, and a small fraction of titanium and aluminum. These steel have very high yield strength of approximately 1800 MPa (18,000 ton/ft^2) and also have excellent fracture toughness. They are used in high temperature applications, such as rocket casings.

Impurities in steels. The common impurities in steels are sulphur, manganese, phosphorous, and silicon. The sulphur content of steels should not exceed 0.05 wt%. It forms FeS along the grain boundaries as a thin film. Since FeS has low melting point, upon heating the metal become brittle, which is called hot shortness. This phenomenon can be reduced by adding manganese (at least five times the weight of sulphur) to the steel. Manganese removes sulphur from the steel by forming globules of MnS. Manganese has significant solute strengthening effect and also reduces the ductile-to-brittle transition temperature. Phosphorous is an effective ferrite strengthener, but the ductility of the steel is affected. Hence, the phosphorous content of steels is generally kept below 0.04 wt%. Silicon is also a good strengthening element of ferrite, but high content of silicon will reduce the ductility of the steel.

9.2.3 Cast Irons

Cast irons are alloys of iron that contain more than 2.11 wt% of carbon. White cast iron contains cementite and pearlite, and it is brittle but has excellent wear resistance. When all the carbon in the cast iron forms graphite, the alloy is called *gray cast iron*. In grey cast iron, the graphite flakes are embedded in ferrite matrix. This cast iron is extremely brittle with only 0.5 percent elongation but has high compressive strength. It has some unique properties, such as excellent fluidity, good wear resistance, and machinability. It is used in internal combustion engine cylinder blocks, locomotives, flywheels, and bases for erection of machinery. In fact, the morphology and size of the graphite flakes significantly influence the mechanical properties. Fine and randomly oriented graphite flakes improve the strength of the iron. Spheroidal graphite (SG) iron contains spherical graphite nodules and is also known as nodular iron or ductile iron. Modifiers such as magnesium or cerium are added to form the nodules. The iron also contains a high silicon content (2.55 wt%), which promotes graphitization during solidification. Spheroidal graphite iron has good tensile strength (400–700 MPa or 4000–7000 fon/ft^2) and elongation (10–18%) and has applications in coke oven doors, gears, and crankshafts.

9.2.4 Stainless Steels

Stainless steels contain chromium (at least 10 wt%) as the main alloying element. These steels are widely used in service conditions where corrosion resistance is critical. The chromium forms a thin protective film (chromium oxide) on the surface of the steel. Nickel and molybdenum are added to improve the chemical and mechanical properties.

Ferritic stainless steels have ferritic microstructure and their chemical composition (wt% chromium $-$ 17 × wt% carbon) percentage number is greater than 12.7. These steels are non-heat treatable; however, the strength can be increased by cold working. AISI 430 stainless steel (16 wt% chromium and 0.12 wt% carbon), which is a ferritic stainless steel, has the following mechanical properties: yield strength (350 MPa or 3,500 ton/ft^2), ultimate tensile strength (550 MPa or 5,500 ton/ft^2), and elongation (30%). Due to the high chromium content, ferritic stainless steels have excellent corrosion resistance. Molybdenum addition (~2 wt%) to these steels containing 18–25 wt% chromium will improve pitting corrosion resistance.

Austenitic stainless steels (300 series), in addition to chromium, also contain nickel, which is an austenite stabilizer. Due to the high cost of nickel, a cheaper austenitic stabilizer, such as manganese, is also used as an alloying element in these steels. Manganese containing austenitic stainless steels form the 200 series alloys. AISI 304 stainless steel, also known as 18/8 stainless steel, which contains 18 wt% chromium, 8 wt% nickel and 0.08 wt% carbon (maximum) is a popular austenitic stainless steel for its attractive mechanical properties: yield strength (240 MPa or 2,400 ton/ft^2), ultimate tensile strength (600 MPa or 6,000 ton/ft^2), and elongation (65%). The formability of these steels is higher than that of ferritic stainless steels. These steels are widely used in chemical industry and household utensils. Molybdenum addition of ~2 wt% improves the pitting corrosion resistance, and the carbon content (<0.03 wt%) improves the intergranular corrosion resistance.

Martensitic stainless steels. In martensitic stainless steels, the chemical composition (wt% chromium $-$ 17 × wt% carbon) percentage number is lower than 12.7. AISI 4010 and 416 are low carbon martensitic steels with 12–14 wt% chromium and 0.15 wt% carbon (maximum). These steels are heat treatable and have the following mechanical properties: yield strength (1200 MPa or 12,000 ton/ft^2), ultimate tensile strength (1300 MPa or 13,000 ton/ft^2), and elongation (5%). Due to their high strength, these steels are used in high-quality ball bearings.

9.2.5 Heat-Treatment Techniques

Heat-treatment techniques are commonly used in industries to tailor the mechanical properties, such as tensile strength, hardness, ductility, and toughness, of metals and alloys. They involve heating/cooling of materials at various temperatures and rates to obtain specific properties. The heat-treatment procedures can be more complex, involving a number of cycles of heating and cooling, to achieve desirable properties. Annealing, aging, quenching, tempering, case hardening, and decarburization are the common heat-treatment techniques.

Annealing is commonly used on metals and alloys after cold working to soften (remove the hardness) or to relieve stress (also called *stress-relieving treatment*). Typically, the material is heated at a specific temperature where the recrystallization process occurs, such that defects caused during cold working are reduced, and then cooled slowly to room temperature. This heat-treatment improves machinability and also enhances electrical conductivity. In ferrous alloys, full annealing forms coarse pearlite. Annealing is also done to produce a uniform microstructure in alloys. Slow cooling in a nonferrous alloy may allow secondary precipitation and consequently improve the strength.

Aging is a heat-treatment procedure where the alloying elements diffuse through the microstructure to form secondary phase particles. The alloys that are strengthened through secondary phase particles, otherwise known as *precipitation hardening*, are called precipitation-hardened alloys. When an alloy is heated to its solutionizing temperature, the alloying elements are in its elemental state and evenly dissolved like in a solution. Upon quenching the alloy, the elements are trapped in the base matrix. However, when the solutionized metal is aged at a certain temperature, nucleation of the secondary phase particles (or precipitates) takes place, followed by the growth of those precipitates and then coarsening. Depending on the size and distribution of the precipitates the strength, hardness and ductility will vary. Basically, aging can occur in two ways: (1) natural aging—precipitation occurs at room temperature, and (2) artificial aging—precipitation occurs at higher temperatures. Stainless steels and some series of aluminum alloys undergo precipitation hardening.

Quenching is rapid cooling of alloys from high temperatures. In ferrous alloys, this process generally increases the hardness, which is by martensite transformation. For quenching of steel, the steel must be heated to above a critical temperature and then rapidly cooled. Cooling can be done by various ways such as air (forced), inert gases, or liquids (water or oil). The chemical composition of the alloy and the quenching method influence the hardness of the material. When the quenching is done at a very high speed, the alloy may undergo cracking. However, in the case of nonferrous alloys, such as aluminum-based, copper-based, and nickel-based, they becomes softer when quenched.

Tempering

Tempering is done on steel to provide some toughness. Generally, untempered martensitc steel is hard and brittle, and tempering will impart some ductility. This heat-treatment procedure consists of heating the steel below the lower critical temperature (205–595 °C) or higher temperature (700 °C), depending on the alloy composition and application.

Case hardening is a thermochemical diffusion process where alloying element, such as carbon or nitrogen, is diffused into the surface of the metal to form an interstitial solid solution. As a result, the surface of the metal becomes harder, and the wear resistance is improved. However, the bulk property, such as toughness of the metal, is not lost. Laser surface treatment is an advanced and popular case hardening treatment due to its high selectivity and versatility for improving the surface properties.

Decarburization. The carbon content of the steel can be altered by controlled heating. The carbon in steel, when austenized and exposed to air (an oxidizing environment) for long periods of time, can slowly diffuse outside and lower its carbon content. This process is called decarburization.

9.3 Nonferrous Metals and Alloys

9.3.1 Copper

Copper is extracted from copper ores, mainly copper sulfides. The ore is mined from large open mine pits as well as through in situ leaching process. Pure copper is orange-red in color, and it does not react with water. However, it reacts with oxygen in the atmosphere to form copper oxide (brown-black color), which protects the copper from further corrosion. On old copper structures, a green layer of copper carbonate can often be seen.

Copper is a ductile metal with high electrical and thermal conductivity. The copper element has one s-orbital electron on top of a filled d-electron shell, which is responsible for the high electrical conductivity. Pure copper is soft, but the hardness can be increased by introducing defects in the crystal lattice, such as grain boundaries. Hence, commercially supplied copper has fine grains, which results in high grain boundaries and better strength. The hardness of copper can also be increased by alloying. Brass (copper-zinc) and bronze (copper-tin) are the two well-known copper alloys used in various applications for many years. Cupronickel, an alloy of copper (90%) and nickel (10%) that is highly corrosion resistant, is used in marine structures as well as in low-denomination coins. Aluminum-containing copper alloys are used in decoration due to their bright golden color. Copper is also added as an alloying element to other metals for improving strength, as in gold and sterling silver ornaments.

Copper and its alloys have been used since ancient times for architecture due to their excellent corrosion resistance. They have been used in roofs, flashings, downspouts, doors, plumbing, and industrial machinery for many years. Copper can be machined and also be welded. Due to its excellent electrical conductivity, copper is mainly used in electrical wires such that it accounts to about 60 percent of its applications. Due to its antibacterial property, copper has found applications in handrails and bathroom fixtures. The demand for copper is increasing as the developed world usage is increasing significantly. Fortunately, copper can be recycled without any loss of quality. It has been estimated that copper (in volume) is the third most recycled metal.

9.3.2 Aluminum

Aluminum is mainly extracted from bauxite ore. Aluminum metal is silvery-white in color, very reactive, ductile, and malleable. It is a lightweight metal with density of about one-third of that of steel. Aluminum can be cast, formed (by drawing and extrusion), and also is machinable. Although the yield strength of pure aluminum is low, aluminum alloys have high yield strengths (200–600 MPa or $2,000-6,000$ ton/ft^2). Alloying elements, such as copper, zinc, magnesium, manganese, silicon, tin, and lithium, are added to improve the strength. Further, certain series of aluminum alloys respond to heat treatments and produce precipitation hardening. The Aluminum Association of America (AAA) classification of aluminum alloys is shown in Table 9.2.

Aluminum is a good electrical and thermal conductor. It is a popular engineering metal due to its low density and resistant to corrosion in normal atmosphere. A thin film of aluminum oxide forms on the metal when exposed to air, which prevents further corrosion. Since oxygen and moisture are impermeable through the thin film, it is attractive for the packing industry (foils and beverage cans). It is also used in construction industry, for example, in doors, windows, and wires. High-strength aluminum alloys are used in automobiles and the aerospace industry, as well as in structural applications. However, one of the limitations of aluminum alloys is the fatigue strength, which is not well-defined in these materials. Aluminum is recyclable, and recycling is a huge industry due to the growing use of aluminum in transportation and the packing industry. Recycled aluminum is known as secondary aluminum.

9.3.3 Magnesium

Magnesium is mainly produced by electrolysis from brine. This is a very energy-intensive process. Among all lightweight metals, magnesium is attractive due to its low density, good mechanical properties and damping capacity, high thermal conductivity, good castability, and electromagnetic shield property. Magnesium alloys are comparable to engineering metals, such as iron-based and aluminum-based alloys, with excellent high strength-to-weight ratio. It is less dense, that is, one-third that

Table 9.2

Classification of aluminum alloys, their major alloying elements, and applications

Series	Major alloying elements	Applications
1xxx	Unalloyed	Sheets, plates, electrical and packing
2xxx	Copper	Aircraft structures
3xxx	Manganese	Buildings, roofing sheets, beverage cans and cooking utensils
4xxx	Silicon	Welding and brazing electrodes
5xxx	Magnesium	Transportation structural plates, marine fittings and aircraft fuel tanks
6xxx	Magnesium, Silicon	Pipes, decorative and architectural structures
7xxx	Zinc, Magnesium, Copper	Aircraft structures
8xxx	Tin, Lithium	Automobiles and aircraft structures

of aluminum and has huge potential in the transportation industry for saving fuel. However, the widespread use of magnesium is hindered due to its poor corrosion resistance. Impurities such as iron, nickel, and copper form secondary phase particles due to their low solubility limits. Since these secondary phase particles are generally electrochemically cathodic to the matrix, they can induce magnesium corrosion. Alloying elements such as aluminium and rare-earth metals have shown to improve the corrosion resistance of magnesium. Magnesium alloys are finding applications in automobile and aircraft industry, as well as in electronic products and sports equipment. Magnesium is also used as an alloying element for making other metal alloys, such as aluminum alloys.

9.4 Types of Failures

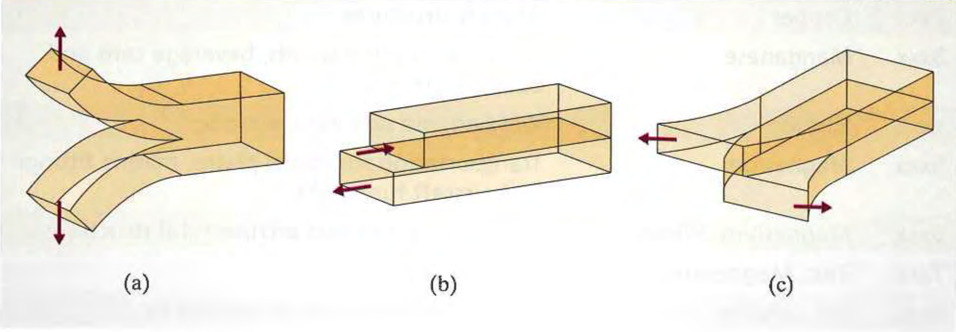
Failure can occur to structural components during service, which can lead to huge economic loss (direct and indirect) and sometimes even be dangerous to human lives. Broadly, material properties and exposure to the environment are the two factors that determine the failure of components. Specifically, microstructure, mechanical loading conditions (tensile/compressive/shear forces), environmental temperature, and aggressiveness of the environment affect the performance of material components in service.

9.4.1 Fracture

Fracture of a material is the breaking of a component into two or more pieces due to mechanical stress. High-purity metals undergo a very large deformation before fracture; for example, pure iron exhibits about 100 percent elongation, whereas an alloy such as cast iron exhibits only about 3 percent elongation. Fracture strength is defined as the stress at which the sample fractures. It is also called the *breaking strength*. There are three ways of applying load for a crack in a material to propagate (Figure 9.3): (1) tensile stress normal to the plane of crack; (2) shear stress acting parallel to the plane of the crack and perpendicular to the crack front; and (3) shear stress acting parallel to the plane of the crack and parallel to the crack front.

Broadly, there are two types of fractures: (1) *ductile fracture*, and (2) *brittle fracture*. In ductile fracture, a significant amount of plastic deformation occurs. The

Figure 9.3 Three types of loading modes: (a) opening mode, (b) sliding mode, and (c) tearing mode.



(Adapted from Raghavan V., 1997)

basic steps involved in a ductile fracture are void formation, crack formation (void coalescence during the necking process), crack propagation, and failure. Plastic deformation will occur when the stress level ahead of the crack tip exceeds the yield strength of the material. Extensive plastic deformation takes place ahead of the crack tip. Nucleation and void coalescence ahead of the crack tip make the crack grow, which results in a dimpled fracture surface. The fracture strength in ductile materials is lower than their ultimate tensile strength. When the material reaches its ultimate tensile strength and the applied load is not increased, the material will still deform until it is fractured. In the case of a brittle fracture, there will be no significant plastic deformation, but at the microscopic level, there is little deformation at the crack tip before sudden fracture. Brittle fractures are commonly seen in covalently bonded materials such as ceramics and glasses. Some metals with body-centered cubic (BCC) and hexagonal close-packed (HCP) crystal structures also undergo brittle fracture at very low temperatures. The fracture strength of brittle materials is generally equivalent to their ultimate tensile strength.

Fractrography is the study of fracture surface to understand the cause for failures. The two common modes of fractures are *intergranular* and *transgranular* (Figure 9.4). Intergranular fracture is when the crack propagates along the grain boundaries. When the crack passes through the grains, it is called transgranular facture. Ductile fracture can be intergranular or transgranular, which depends on the crack propagation path. A typical ductile fracture is a *cup-and-cone shaped failure* surface with necking (reduction in cross-section area), whereas brittle fracture is relatively flat without necking (Figure 9.5). Brittle fractures are often intergranular since grain boundaries have a surface energy, and hence it is an easy pathway for the crack to propagate along the grain boundaries. Cleavage fractures can occur in brittle materials, that is, when tensile stress acting normal to crystallographic planes with low bonding.

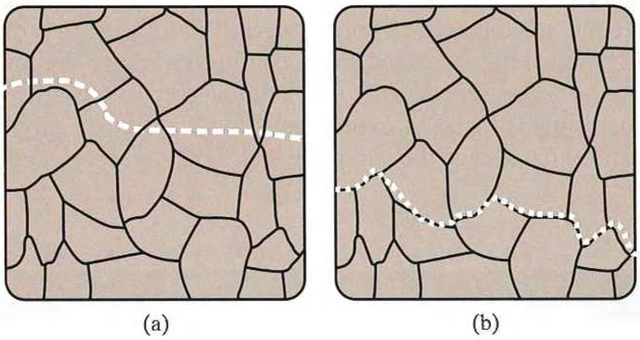
Ductility of a material is measured using a tensile sample in two ways: (1) percent elongation (E), that is, the change in length by the original length of the sample; and (2) percent reduction in area (RA), that is, comparing the original cross-sectional area of the sample to the final cross-sectional area:

$$\%E = 100\,(l_f - l_o)/l_o \tag{9.1}$$

$$\%RA = 100\,(A_o - A_f)/A_o \tag{9.2}$$

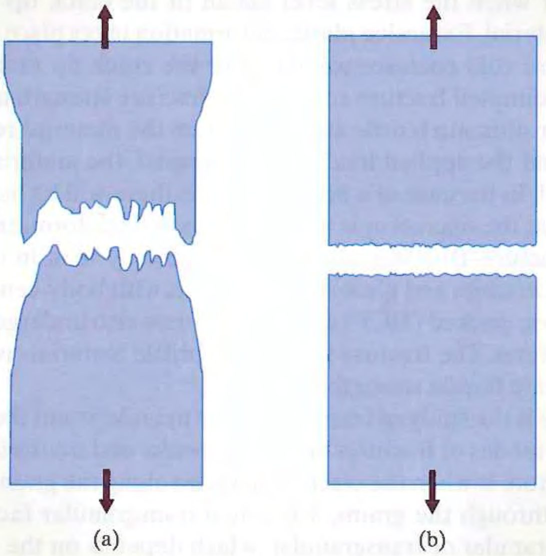
The two modes of fractures: (a) intergranular and (b) transgranular.

Figure 9.4



(Note: white broken lines are the crack propagation path.)

Figure 9.5 Types of fractures: (a) ductile fracture and (b) brittle fracture



where l_o is the original length of the sample, l_f is the final length of the sample after fracture, and A_o and A_f are the original cross-sectional area of the sample and the final cross-sectional area at the location of the fracture, respectively.

Fracture toughness is defined as the ability of a material to resist fracture when containing a crack. The fracture toughness is very low in materials which undergo brittle failure. For design applications, fracture toughness is an important factor. It is represent by K_{IC} ("I" is mode 1 or uniaxial loading, and "C" is the critical load), which is the critical stress intensity factor at the crack tip needed to cause catastrophic failure under simple uniaxial load. Fracture toughness can be calculated using the following equation:

$$K_{IC} = Y\sigma_f\sqrt{\pi a} \tag{9.3}$$

where Y is a dimensionless geometry factor of the order of 1, σ_f is the overall applied stress at failure, and a is the surface crack length. The unit for fracture toughness is $\text{MPa}\sqrt{m}$. K_{IC} is associated with plane strain conditions, where the sample thickness is relatively large as compared with the notch dimension. In the case of thin samples (plane stress conditions), the fracture toughness is a function of sample thickness. In general, when the thickness of the sample is $\geq 2.5\ (K_{IC}/\text{yield strength})^2$, then plane strain exists.

EXAMPLE 9.1

A metallic material has a yield strength of 1000 MPa. Determine the size of a surface crack which leads to fracture when the applied stress is 600 MPa. The fracture toughness (K_{IC}) of the materials is 150 MPa $m^{0.5}$

$$K_{IC} = \sigma_f\sqrt{\pi a}$$
$$K_{IC} = 150\ MPa\sqrt{m}$$
$$\sigma_f = 600\ MPa$$
$$a = ?$$
$$150 = 600\sqrt{3.141 * a}$$
$$a = 19.8\ mm$$

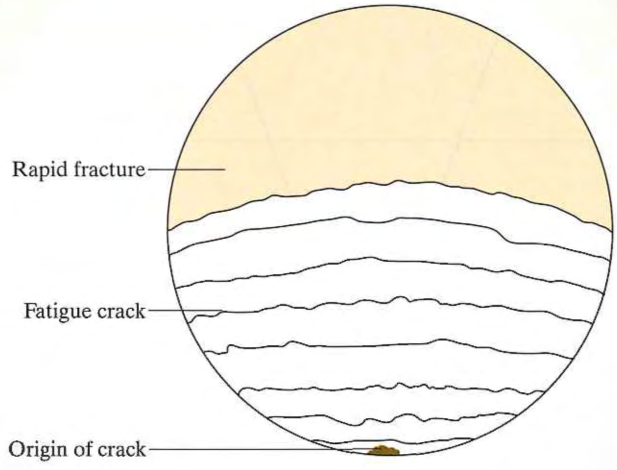
Ductile-to-Brittle Transition. A material which is ductile under one condition can become brittle in another condition, which is called ductile-to-brittle transition. For example, hexagonal close-packed (HCP) and body-centered cubic (BCC) materials are ductile at high temperatures and become brittle at low temperatures. The ductile-to-brittle transition temperature is dependent on the density of secondary phase particles in the alloy. As the number of these secondary phase particles increases, the ductile-to-brittle transition temperature also increases. Further, the shape of the particles also has an effect. Segregation of impurities along the grain boundary can reduce the strength of the boundary and, as a result, increase the ductile-to-brittle transition temperature.

9.4.2 Fatigue

Fatigue is a material failure phenomenon due to cyclic loading to a stress level below the ultimate tensile strength of the material. Generally, structural materials experience tension (pull) and compression (push) during service, which causes the cyclic loading. A material can withstand a stress up to its ultimate tensile strength when it is a single loading. However, when the loading is repeated, the material may fail at a stress level lower than that of its ultimate tensile strength. Cyclic stress can cause microscopic plastic deformation even when the stress is lower than the yield strength of the material. When the loads are above a critical threshold stress, microscopic cracks form at the surface or grain interfaces which are the stress concentrators, and they ultimately reduce the load-bearing capacity of the material (Figure 9.6). The

Various regions in a fatigue-fractured surface

Figure 9.6



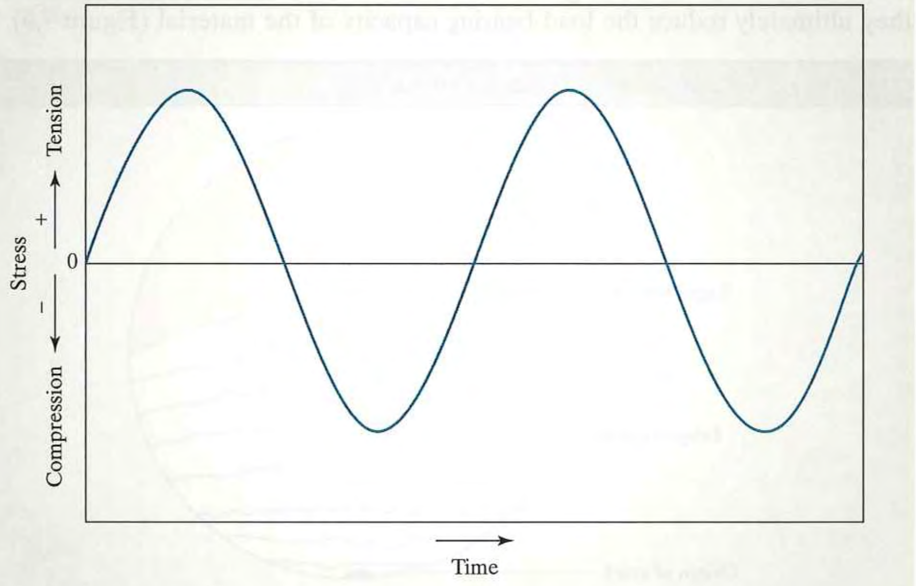
(Adapted from Raghavan V., 1997)

micro-cracks will reach a critical size and propagate quickly to cause the material to fracture. Fatigue life is also affected by the shape of the component. For example, sharp corners in a component will reduce fatigue life, whereas round holes or smooth surfaces will increase fatigue strength and life.

In fatigue strength measurement, a cylindrical tensile sample is rotated by a motor under a specific load. The upper surface of the sample experiences tension, and the lower surface, compression. The sample surface passes through a full cycle of tension and compression during each revolution (Figure. 9.7). The cyclic loading can be of three types: (1) tension–tension, (2) tension–zero, and (3) tension–compression. The maximum and minimum stresses are tensile in nature for the tension–tension type of loading. The minimum stress is zero in the tension–zero loading, and it is compressive in the tension–compression loading. The stress amplitude is half the difference between the maximum and the minimum stress levels.

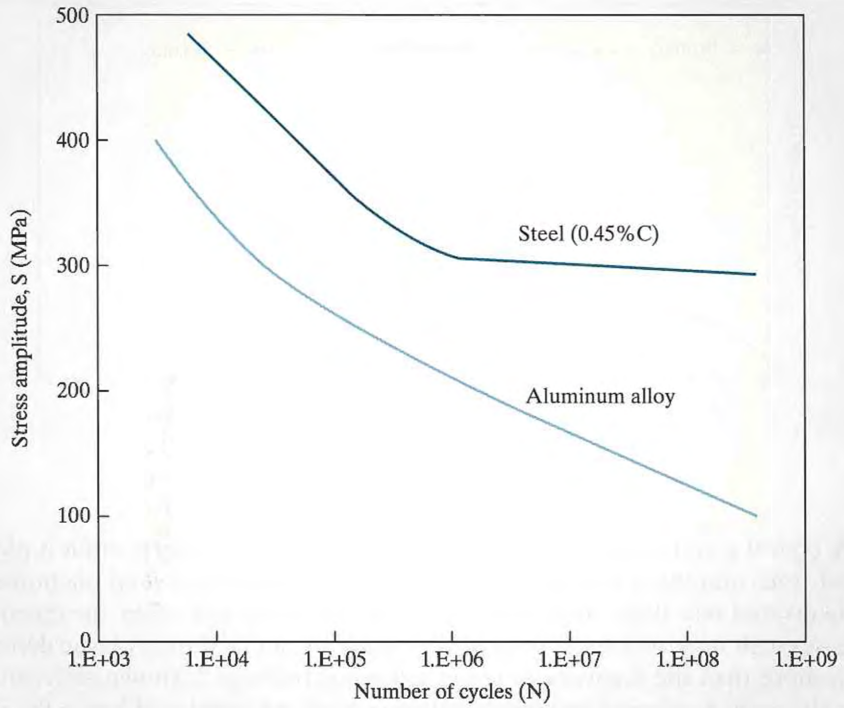
A typical fatigue curve, also known as S-N curve, is shown in Figure 9.8, where the plot is between stress (S) versus the number of cycles (N). The fatigue strength of a material is lower than its ultimate tensile strength and can be even lower than its yield strength. The decay in strength with increasing number of cycles reaches a limit, which is called the fatigue strength or *endurance limit.* It is a characteristic of ferrous alloys, but nonferrous alloys generally do not exhibit such distinct limit. Steel exhibits a distinct fatigue limit such that when the stress amplitude is below a certain value, failure does not occur. For aluminum alloys, there is no distinct fatigue limit; the fatigue strength is specified by N value. Ceramics also undergo fatigue failure but under static loading. The fatigue mechanism in metals is mechanical, but in ceramics it is chemical in nature predominately around room temperature. Fatigue in polymers is similar to that in metals.

Figure 9.7 A typical fatigue cycle with tension and compression stresses



Fatigue S-N curve for a steel and an aluminum alloy

Figure 9.8



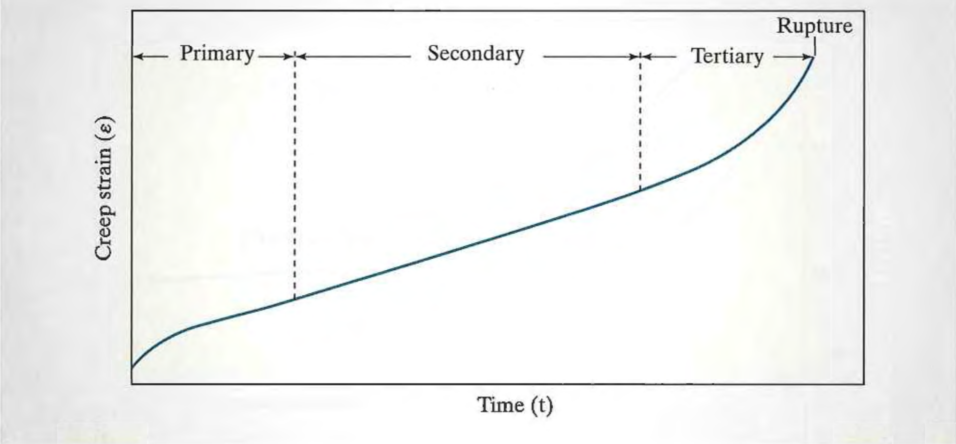
(Adapted from Raghavan V., 1997)

Factors that affect fatigue life are (1) frequency of cyclic loading; (2) forms of stress cycles; (3) environment (inert or corrosive); (4) temperature; and (5) thickness of the sample. Mechanical deformation such as cold working can increase fatigue resistance. Fatigue life can also be increased by preventing or delaying the initiation of cracks at the surface. Some of the methods to increase fatigue life are (1) shot peening, that is, introducing compressive residual stress on the surface by repeated impingement of hard balls; (2) fine grain size; and (3) a polished surface to minimize the chances of providing stress concentrate sites.

9.4.3 Creep

Creep is a phenomenon of plastic deformation and failure of metals at high temperature and tensile stress. Creep is temperature and time dependent and occurs at a temperature above 0.4 Tm (where Tm is the melting point of the metal in K) and under a long-term exposure of high levels of tensile stress, which are still below the yield strength. Creep-resistant materials are needed for applications of structural and machine components at high temperatures. The materials should be able to withstand high temperatures without undergoing creep, since this phenomenon will cause dimensional changes that are not generally accepted in the design. The deformation rate is dependent on the materials properties, applied load, exposure time, and temperature. Since creep is a time-dependent deformation, fracture (brittle fracture or sudden failure) is not necessary for the failure of the component.

Figure 9.9 A typical creep failure curve showing three stages in the failure mechanism



A typical creep curves is shown in Figure 9.9, where the creep strain is plotted against time, and the load and temperature are constant. The creep phenomenon can be divided into three stages. In stage 1, known as *primary creep*, the creep rate decreases with time, where the work hardening of the metal during plastic deformation is more than the recovery or creep softening. In stage 2, known as *secondary creep*, the work-hardening and creep-softening rates are equal, and hence the slope of the strain-time curve is constant. Stage 3, *tertiary creep*, is an accelerated rate of creep, where necking occurs and leads to fracture. In this stage, voids in the material form and coalesce to cause intergranular fracture. Recrystallization and coarsening of precipitates also contribute to tertiary creep. If the load or temperature is increased, the three stages will shift to shorter times. Creep curves at different temperatures are useful for selection of materials for high-temperature applications, where typically, creep data is critical in design and material selection.

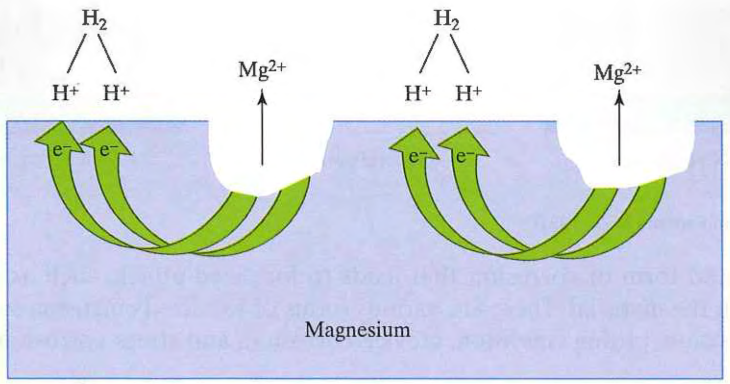
All materials will undergo creep when the temperature is near to its melting point. Materials with high melting points have better creep resistance. Metals and plastics with low melting points can creep at ambient temperatures. For example, lead (Pb) creeps at ambient temperatures, but copper (Cu) and iron (Fe) need a higher temperature for creep-related failures. Precipitation-hardened materials are useful in providing creep resistance, but the particles should not coarsen during service, which would have a detrimental effect on the mechanical properties. Moderate creep in concrete can be beneficial since it relieves tensile stresses that could potentially lead to cracking.

9.4.4 Corrosion

Corrosion is defined as material deterioration due to chemical reaction with an environment. The consequence of corrosion is loss of material thickness, surface appearance, and mechanical properties. Engineering metals and alloys generally corrode at varying degrees in aqueous environments during service. A common example of corrosion is rusting of iron or steel. It has been estimated that the loss due to corrosion in industrialized countries is about 3 percent of the GNP (gross national product).

Electrochemical corrosion of magnesium in aqueous environment

Figure 9.10



The loss can be direct, such as replacement cost, as well as indirect loss, such as shut down of machinery for replacement which leads to productivity loss.

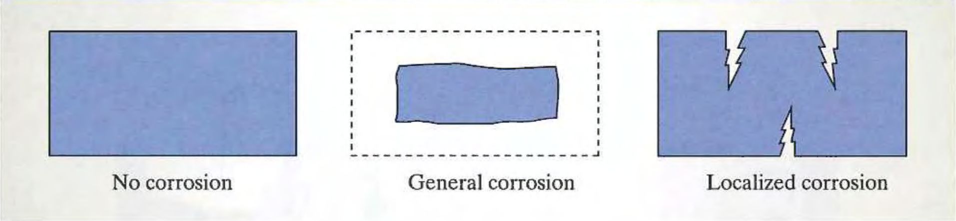
Corrosion is an electrochemical oxidation of metal in an aqueous environment (Figure 9.10). It involves two types of reactions: anodic reaction (for example, $Mg \rightarrow Mg^{2+} + 2e^-$) and cathodic reaction (for example, $2H^+ + 2e^- \rightarrow H_2$). Although corrosion particularly occurs in metals, other materials such as ceramics or polymers, also undergo corrosion, commonly termed as degradation. Corrosion affects properties of materials and structures, including strength and appearance. Materials may corrode when exposed only to moist air, but the rate of corrosion can be affected by the other chemicals present in the exposed environment.

Passivation in metals and alloys is a phenomenon which reduces the corrosion rate of materials. For example, when an iron rod is exposed to dilute nitric acid, the metal will corrode rapidly. However, when the same iron rod is exposed to concentrate nitric acid, it quickly forms a passive film, and the corrosion of the metal ceases. In this case, the iron is said to become passive, meaning that it is immune to corrosion. The difference in the behavior can be explained by the kinetics of oxidation in the two acids. Concentrated acid effectively oxidizes the iron to form a thin protective film, whereas the dilute acid is not strong enough to produce the passive film and hence, the rod undergoes continuous corrosion. Another classic example is aluminum, which is basically an active metal, but does not undergo corrosion in a normal atmosphere. When aluminum is exposed to air, a thin film of aluminum oxide forms immediately on the surface of the metal which protects it from further corrosion. Basically, there are three methods to enhance passivity in metals: (1) the application of an external current, also called as *impressed current* to accelerate the oxidation process, which passivates the metal in a method called *anodic protection*; (2) the addition of passivators, for example, oxidizing ions such as nitrates and chromates; and (3) the addition of alloying elements to enhance passivation, such as the addition of chromium and nickel to steel, which is stainless steel.

Types of Corrosion

Corrosion can be broadly classified as general and localized corrosion (Figure 9.11). General corrosion, also called *uniform corrosion*, occurs across a wide area nearly uniformly and leads to loss of thickness of the material. Localized corrosion is a

Figure 9.11 Schematic representation of general corrosion and localized corrosion



(Adapted from Fontana M.G., 1987)

concentrated form of corrosion that leads to localized attack, such as pitting or cracking in the material. There are various forms of localized corrosion such as galvanic corrosion, pitting corrosion, crevice corrosion, and stress corrosion cracking (SCC).

Galvanic corrosion occurs when two different metals are in contact (physical or electrical) and exposed to a corrosive environment. The most active metal (anode) corrodes quicker, and the more noble metal (cathodes) is protected or corrodes at a very low rate. However, if the metals are not connected, then they corrode at their own rate, such that the active metal will corrode at a slower rate than when connected to a noble metal. The activity of the metal can be determined from the electrochemical potentials (Table 9.3). For example, magnesium is an active metal as compared to copper. Galvanic corrosion is a major problem in many industries, including the marine industry, when different materials in structures are in contact with corrosive environments. The phenomenon of galvanic corrosion can also be advantageously used for corrosion protection. For example, zinc is used as a sacrificial anode to protect steel structures; here zinc acts as an anode and the steel structure becomes a

Table 9.3 Electrochemical potentials of pure metals

Noble Metal

Metal	Potential (V)
Au (Gold)	+1.43
Ag (Silver)	+0.80
Cu (Copper)	+0.34
Pb (Lead)	–0.13
Sn (Tin)	–0.14
Ni (Nickel)	–0.25
Cd (Cadmium)	–0.40
Fe (Iron)	–.044
Zn (Zinc)	–0.76
Ti (Titanium)	–1.63
Al (Aluminum)	–1.66
Mg (Magnesium)	–2.37
Na (Sodium)	–2.71
K (Potassium)	–2.93
Li (Lithium)	–3.05

Active Metal

Galvanic corrosion—effect of electrochemical potential of the coating material

Figure 9.12

(Adapted from Fontana M.G., 1987)

cathode, whereas when zinc is replaced by tin (a noble metal), it would lead to localized corrosion in the steel structure (Figure 9.12). Galvanic corrosion is influenced by the size of the anode, the type of metal, and the environment. The surface area ratio of the anode and cathode has a significant effect on the corrosion rate of the material. A large cathode and a small anode in a material will cause localized corrosion.

Pitting Corrosion

Pitting corrosion generally occurs in the presence of chloride ions, which can affect the passive film on metal or alloy surface. Chloride ion attack is localized, and corrosion can be greatly increased locally, leading to pit formation. These pits can grow due to an autocatalytic process. They can act as stress raisers and lead to another form of localized corrosion known as stress corrosion cracking (SCC) in the presence of tensile stress. Pitting is basically depassivation of a small area that becomes the anode (corroding part), and the surrounding area becomes the cathode, producing a condition for galvanic reaction. A pit can grow deep inside the material and become difficult to detect due to its small width and can also be covered by corrosion products.

Crevice corrosion is another form of localized corrosion that occurs in confined spaces or crevices where, typically, the oxygen concentration in the liquid is lower as compared to the outer liquid. This leads to a differential aeration cell, causing corrosion inside the crevices. Crevices are small gaps or contact areas between parts like inside cracks, under gaskets or seals, and in spaces under deposits. Factors such as crevice geometry, material type, and environment can influence crevice corrosion.

Stress corrosion cracking (SCC) is a failure mechanism of a material caused due to the synergistic effect of tensile stress and presence of corrosion environment. This form of corrosion can lead to catastrophic failures. A ductile material (normally with high strength), when exposed to tensile stress and a corrosive environment, can fail without undergoing any significant deformation; hence, the failure is sudden. High-strength steels, stainless steels, and aluminum alloys undergo SCC in chloride-containing environments. SCC can be broadly classified into two types: (1) anodic dissolution–assisted cracking, and (2) hydrogen-induced cracking. In anodic dissolution–assisted cracking, the metal dissolution at the crack tip would lead to crack propagation and failure. In principle, anodic dissolution is accompanied by cathodic reaction. When the cathodic reaction is a hydrogen-evolution reaction, depending on the material, the hydrogen can diffuse inside the material and cause embrittlement. This phenomenon is called hydrogen embrittlement. Mechanistically, the hydrogen can chemically react and form hydrides or occupy trapping sites and cause localized plasticity during deformation.

Steel bars corrosion. Corrosion of steel is one of the most common causes of bridge accidents, for example, Mianus River Bridge (Connecticut, US) in 1983. Severe structural problems can occur due to corrosion of concrete-covered steel or iron. In fact, this is one of the most common reasons for failure of reinforced concrete bridges. Hydration of cement leads to an alkaline environment (pH ~13), in which reinforced steel bars form a thin protective oxide film which prevents further corrosion. However, the protective film can be destroyed by concrete carbonation or the presence of chloride ions. Corrosion products in reinforced steel bars (for example, Fe_3O_4, $Fe(OH)_2$, $Fe(OH)_3.3H_2O$) can occupy about two to six times greater volume than the original steel bar. As a result, tensile stresses can be generated and crack the concrete, resulting in delamination. Localized corrosion of steel bars can reduce the load-bearing capacity and also affect the fatigue strength.

Basically, there are two stages of corrosion in the service life of reinforced-concrete structures. In the first phase, the passive film is lost due to carbonation or penetration of chloride ions. This is also called the initiation stage, where the carbon dioxide and chloride ions penetrate gradually toward the steel rebars. The alkaline pH environment, which protects steel rebars, can be neutralized by carbon dioxide and thereby affect the passive film. Similarly, chloride ions can penetrate from the environment into the concrete. The rate of ingress of the chloride ions depends on the porosity and permeability of the concrete. The second stage is when the corrosion propagates and causes damage to the structure. Once the protective passive layer is affected, the corrosion initiates and propagates. Carbonation completely destroys the passive layer, whereas chloride ions would cause localized corrosion. Hence, general corrosion occurs with carbonation, and pitting corrosion occurs in the presence of chloride ions. Corrosion in structures can also be affected by DC stray current, which could destroy the passive layer of the steel bars.

Corrosion Rate Measurement

The weight-loss method is a simple and cheap method to measure the corrosion rate of materials. The material is exposed to the corrosive environment for a specific period of time, and the weight loss is monitored. The corrosion rate is calculated using the following equation:

$$\text{Corrosion rate in mpy} = kW/(\rho At) \tag{9.4}$$

where k is constant (534), W is weight loss (mg) of the metal in time t (hours), A is surface area of the metal exposed (inch^2), and ρ is the density of the metal (g/cm^3). The unit for corrosion rate is mpy, (mils per year).

Corrosion Prevention Methods

A simple method for corrosion protection of materials is surface coating, where a thin layer of polymer coating such as paints, ceramic coating, or metal plating is applied on the metal. These coating materials provide a barrier between the metal and the corrosive environment. Painting polymeric materials such as polyurethane is relatively easy, cheap, and quick. This can be done using a brush or spray method. In metal plating, choosing the right metal is critical, because failure in a small area of the coated

EXAMPLE 9.2

A steel plate corroded at a rate of 200 mpy when immersed in seawater. The original surface area of the plate was 25 cm^2, and the material weight loss after corrosion was 1.5 kg. Calculate the time of immersion. (Note: the density of steel is 7.9 g/cm^3)

$$\text{Corrosion rate (mpy)} = kW/(\rho At)$$

$$t = \frac{KW}{\rho A(CPR)}$$

$$A = 25\ \text{cm}^2 * \frac{1\ \text{inch}^2}{6.4516\ \text{cm}^2} = 3.875\ \text{inch}^2$$

$$t = \frac{(534)(1.5 \times 10^6\,\text{mg})}{(7.9\text{g/cm}^3)(3.875\ \text{in.}^2)(200\ \text{mpy})}$$

$$t = 130829\ \text{h} = 14.9\ \text{yr}$$

surface can affect the protection of the metal. This could lead to galvanic corrosion when the coated metal is noble to the substrate. For example, in chromium coating on steel, when there is a coating defect, the cathodic chromium will intensify the corrosion of the exposed area of the steel. However, an active metal coating such as zinc on steel, commonly known as galvanization, would not have a deleterious effect. Chemical surface treatment is another method of enhancing the corrosion resistance of metallic materials. Anodizing is a surface treatment method where a thick passive film is created on the surface of the metal in an electrochemical bath.

Inhibitors are small amounts of substances (or additives) added to the environment that reduce the rate of corrosion. Inhibitors basically slow down the rate of oxidation or reduction reactions. There are two types of inhibitors: (1) *anodic inhibitors* (otherwise known as passivators), which promote the formation of passive film on the metal surface; and (2) *cathodic inhibitors*, which reduces the cathodic reactions such as the hydrogen evolution reaction and water reduction reaction.

Cathodic protection is a well-known technique commonly used to protect steel structures, such as fuel pipelines and tanks, by making them cathodic. It is achieved by employing a sacrificial anode or by external impressed current. This method can be applied only in moist soils or aqueous media since a conducting media is required for passage of current. In the sacrificial anode method, a material, an anode which corrodes quicker than the material of the equipment or structure, is connected. Here, the anode will corrode and protect the equipment/structure, which is made cathodic. In the case of impressed current method, the equipment/structure is held at a cathodic potential, so that it does not corrode. Anodic protection is used for metals that exhibit passivity (for example, stainless steel). The passive current of the metal should be small over a wide range of potentials. Contact between dissimilar metals, for example, a steel shaft attached to copper tubes, will lead to corrosion. Hence, avoiding such situations in design would prevent corrosion. The design should be such that the anode part of the structure is larger than the cathode since small anodic areas means a high rate of corrosion, owing to high current density.

9.5 Summary

1. Steels are versatile engineering materials. They have good strength, ductility, and formability. Steels are broadly classified into four types: (1) low carbon steel, (2) mild steel, (3) medium carbon steel, and (4) high-strength steel.
2. Cast irons have high carbon content (>2.11 wt%) and possess excellent fluidity and good wear resistance but exhibit poor ductility.
3. Stainless steels contain chromium as the main alloying element, which improves the corrosion resistance. The main three types of stainless steels are ferritic, austenitic, and martensitic stainless steels.
4. Heat-treatment techniques can be used to tailor the properties of many engineering alloys such as steels, stainless steels, and some series of aluminum alloys.
5. Nonferrous metals and alloys such as copper-based, aluminum-based, and magnesium-based alloys have widespread applications due to their attractive physical and mechanical properties.
6. Fracture is a separation of a component into two or more pieces due to mechanical loading. The degree of plastic deformation determines whether a material is ductile or brittle. Generally, metals, alloys, and polymers are ductile, whereas ceramics are brittle.
7. Cyclic loading (tension and compression) can cause failure in materials at a stress level lower than that of their ultimate tensile strengths, and the phenomenon is known as fatigue.
8. Creep is a high-temperature deformation failure mechanism. Materials having a high melting point have better creep resistance.
9. Materials deterioration due to chemical reaction with the exposed environment is called corrosion. General and localized corrosion are the two broad types of corrosion. Corrosion can be prevented by methods such as coating, chemical surface treatment, cathodic/anodic protection, and the use of inhibitors in the environment.

Exercises

1. A high-strength steel has cracks not more than 5mm in length. The fracture toughness (K_{IC}) of the materials is 100 MPa $m^{0.5}$. Determine the maximum service stress of the steel.
2. A high-strength metal rod fractures at a stress level of 1500 MPa. Calculate the size of the crack that caused the failure of the rod. The fracture toughness (K_{IC}) of the high-strength metal is 200MPa $m^{1/2}$.
3. A 10 mm thick steel sheet of area 250 cm^2 was exposed to seawater. The steel sheet was found to experience a weight loss of 300 g due to corrosion. Calculate the corrosion rate in mpy.
4. Explain why chromium in stainless steels makes them corrosion resistant in seawater.
5. Why are ferrous alloys more widely used in engineering application than non-ferrous alloys?
6. Compare steel and cast iron in terms of microstructure and chemical composition.
7. What is the difference between hypoeutectoid and hypereutectoid steels?

8. Give a situation where cast iron is preferred over steel.
9. Explain the difference between annealing, aging, and tempering.
10. Suggest a method to reduce hot shortness in steels.
11. Explain the role of vanadium and tungsten in high carbon steels.
12. Explain the difference between intergranular and transgranular fractures.
13. What are the three types of reaction in an iron–carbon diagram?
14. What are the common impurities in steels?
15. Briefly explain ductile-to-brittle transition.
16. What is the purpose of case hardening?
17. Suggest a method to increase the fatigue life of components.
18. Explain the influence of load and temperature on the creep behavior of metallic materials.
19. Explain the mechanisms of concrete delamination due to corrosion.
20. What is the difference between anodic and cathodic protection of metallic structures?

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CHAPTER 10

Sydney Harbour Bridge is a steel arch bridge with a total steelwork weighing 52,800 tonnes. Courtesy of R. Tuladhar, James Cook University, Australia

Steel

10.1 Introduction

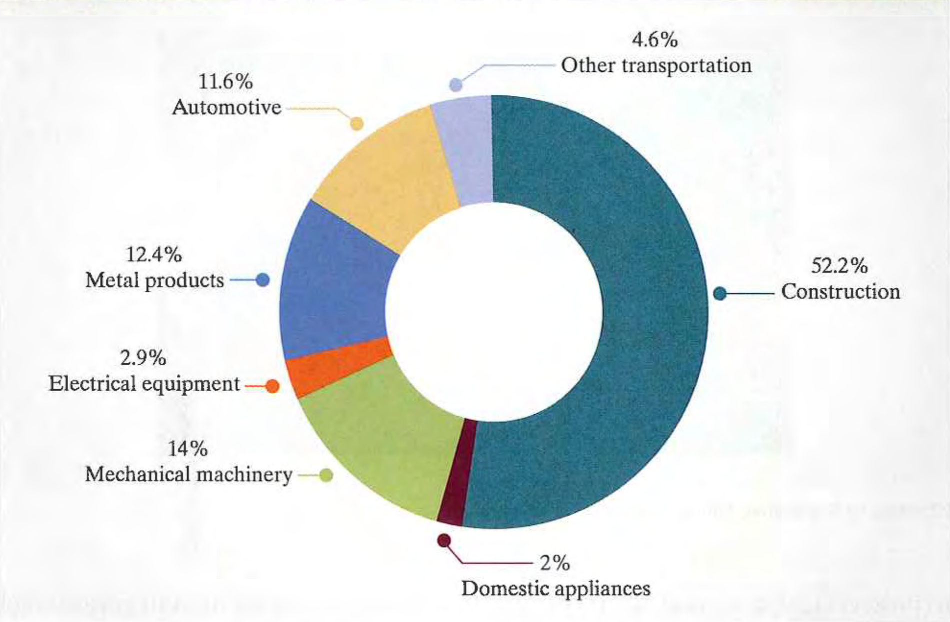
Steel is an essential part of our modern civilization. Due to its excellent strength-to-weight ratio combined with its versatility, durability, and ductility (ability to withstand large deformation without fracture), it forms a critical component of construction, machinery, and manufacturing industries worldwide. As of 2013 data, 1.6 billion tonnes of steel are manufactured every year around the world. More than 50 percent of the total steel produced goes into buildings and infrastructures (Figure 10.1) (WorldSteel, 2014). Structural steel used in the construction industry is produced from iron ore, and it is an iron alloy containing carbon (ranging from 0.2 to 1.5% by weight) and other elements. This range of carbon content in steel gives it its unparalleled strength, hardness, and ductility. With the advent of modern steel manufacturing processes like basic oxygen steelmaking (BOS) and the electric arc furnace (EAF), steel production efficiency has vastly improved, and steel has become one of the most efficient and versatile construction material for building skyscrapers to bridges.

10.2 Advantages of Steel

Steel has numerous advantages over other construction materials. One of the main advantages of steel is its high strength-to-weight ratio, and because of that, smaller steel sections can be used in a structure, reducing the overall dead load (self-weight) of the structure. Therefore, it is the preferred construction material for high-rise buildings and long-span bridges. With the use of slender steel columns and beams, building space can be optimized, increasing the productivity of the structure. Steel also has a high stiffness and high ductility compared to other construction materials. The ductility of a material is its property to sustain large deformation without failure. Steel is a quite ductile material, making it a suitable construction material for high

Steel use in construction

Figure 10.1



(Based on WorldSteel (2014). Steel Solutions in the Green Economy, World steel association.)

seismic areas. Concrete structures also need to be reinforced using steel to take the tensile stresses in concrete and to make it more ductile. Furthermore, steel structures are termite- and rot-proof unlike timber structures. Steel also is a homogenous material and has uniform and predictable material properties. Unlike concrete, tension and compression properties of steel are similar.

Furthermore, steel sections are fabricated at a factory, which gives excellent quality control and tighter construction tolerances compared to other construction materials. The speed of erecting a steel structure is much faster than using other materials. Steel is produced in a number of shapes and sizes, making it one of the most versatile construction materials. Additions or extensions to existing steel structures are also much easier. Steel construction also does not need expensive formworks like that for concrete.

Steel has inherent sustainability benefits as it is 100 percent recyclable and has a high scrap value. It provides a healthy living environment for the occupants. Furthermore, steel structures can be repaired easily, minimizing the downtime of the structure. Steel structures most often can be easily dismantled and reerected again at a different location. With these numerous advantages, steel is widely used as construction material all around the world. A very good example of steel structure is the Sydney Harbour Bridge in Sydney, Australia (Figure 10.2).

10.3 Limitations of Steel

There are some inherent limitations in steel, which should be taken into consideration while designing steel structures. Properties of steel can drastically degrade at high temperatures or during a fire. Hence, it is important to take necessary measures

Figure 10.2 Sydney harbour bridge, Sydney Australia

(Courtesy of R. Tuladhar, James Cook University, Australia)

to protect steel structural members from fire. Concrete is often used to encase steel members to protect them from fire damage (Figure 8.2). Steel is also susceptible to corrosion which can lead to loss of its strength, resulting in serious damage to the structure. A protective coating or paint is applied to prevent corrosion in steel structures. Strength of steel may also have an undesirable brittle failure (abrupt failure without warning) at low temperatures. A large number of stress reversals can also lead to reduced strength in steel members.

As pointed out earlier, steel structures, due to steel's high strength-to-weight ratio, are generally slender compared to timber or concrete structures. Buckling failure (instability leading to sudden failure of the member) can be critical in slender steel members subjected to compression and bending. Proper lateral resistance and bracing need to be provided to prevent buckling in slender steel beams and columns.

10.4 Iron- and Steel-Making

Iron ore forms the 5 percent of the Earth's crust and is the fourth most abundant element after oxygen, silicon, and aluminium. Iron ores are composed of iron oxide minerals such as hematite (Fe_2O_3), goethite ($FeO(OH)$), limonite ($FeO(OH)\cdot nH_2O$), and magnetite (Fe_3O_4). The history of using iron in human civilization dates back to 2000 BCE, to the beginning of Iron Age. In ancient times, iron ore was heated in a charcoal fire in bloomery furnaces. The charcoal reacts with oxygen, generating first carbon monoxide and then carbon dioxide. This process produces a spongy, porous mass of iron with low-carbon content (0.02 to 0.08%, absorbed from charcoal) and entrapping high densities of impurities (slag). The impurities were removed by heavy hammering to produce iron with very low carbon content known as wrought iron. Wrought iron is tough and workable. Most of the metal produced and used during the Iron Age was wrought iron.

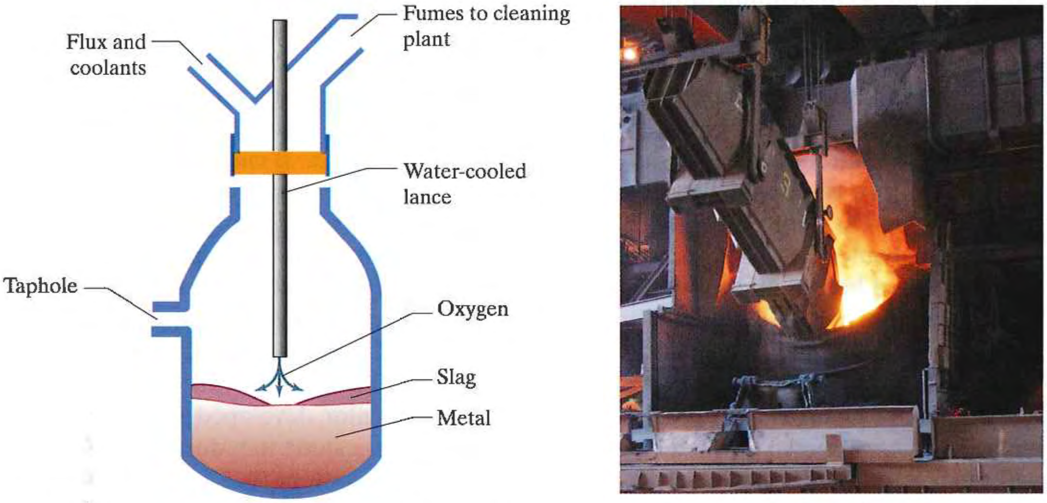
Production of iron in ancient times was severely limited due to the inability of achieving high temperature in bloomery furnaces. In the eighteenth and early nineteenth centuries, blast furnaces were invented in Europe, which made it possible to heat iron ore at very high temperatures and produce iron in large quantities for industrial use. At high temperatures, iron absorbs carbon rapidly, and the higher carbon content lowers its melting point, resulting in molten and castable iron. Reducing substances (also known as flux), such as limestone, facilitates the removal of impurities from the iron ore. The process of melting the metal out of its ore and using suitable reducing substances to produce metal is known as smelting. Molten iron is directly cast into molds to produce desired shapes and, hence, is called cast iron. The molten iron can be poured to produce an intermediate product known as pig iron (so named as the iron ingots cast resembled a sow and suckling piglets). With high carbon content (3–4.5%), cast iron is very hard and brittle and is not malleable. Iron produced before mid-nineteenth century was mostly cast iron and was used in construction, especially in making rails. In the eighteenth century, charcoal was replaced by coal in the iron-making process as the use of charcoal caused massive deforestation (Ashton, 1968).

In 1856, Sir Henry Bessemer, a British engineer and inventor, developed a method where hot air was blown through the molten pig iron and limestone to reduce carbon and other impurities to produce steel with reduced carbon content (0.2–1.5%). This process of steel production was known as the Bessemer process. With the reduced carbon content, steel gained its ductility and hardness that made it suitable for the use as construction material. By the late nineteenth century, the Bessemer process was largely replaced by the open-hearth process, invented by German engineer Karl Wilhelm Siemens, which could produce steel with low nitrogen and allowed precise quality assessment. This process used a shallow, open-hearth furnace where solid or molten pig iron and scrap metal were melted with limestone. This production process was however, slow and required external heat input. Steel used for construction of rails, bridges, machines, and vehicles in the nineteenth and twentieth century was produced through the Bessemer and Siemens processes. By the late twentieth century, the open-hearth process was replaced by more efficient Basic Oxygen Steelmaking (BOS) process and electric arc furnace (EAF) processes. In modern steelmaking, BOS and EAF together account for 98 percent of the total steel produced in the world (Krauss, 2005).

10.4.1 Basic Oxygen Steelmaking (BOS)

The BOS process is an adaptation of the Bessemer process where the pig iron and scrap is converted into steel by blowing pure oxygen (instead of air like in the Bessemer process) over the molten pig iron produced in a blast furnace. The oxygen blown into the furnace, instead of air, reduces impurities. Due to the simplicity and flexibility of the process, BOS is now the main bulk production process used around the world for refining iron ore into steel. The BOS process also uses 25–35 percent steel scrap (recycled products made of steel) to make new steel. Scrap metal and molten pig iron from the blast furnace are charged (the process of feeding ingredients into the furnace) into the BOS vessel (Figure 10.3). The proportion of charge (molten pig iron and flux) is controlled precisely to produce quality steel. Pure oxygen at high pressure is then blown at supersonic speed on the molten metal through a water-cooled vertical lance. Oxygen combines with carbon and other impurities to purify steel. These oxidation reactions are exothermic (generate heat), which increase the temperature of the furnace, and hence, no external heat is required in the process. Lime-based fluxes are added which removes the impurities in the form of slag. The BOS converter is then tilted to pour the steel into a steel ladle. This process is called tapping the steel.

Figure 10.3 Basic oxygen steelmaking

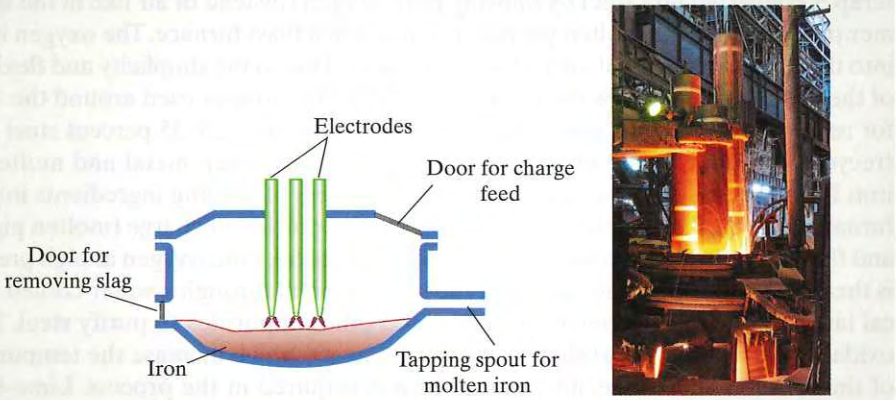


(Courtesy of S. Kelly, OneSteel Whyalla Steelworks, Australia)

10.4.2 Electric Arc Furnace (EAF)

The EAF process essentially uses scrap steel to produce new steel. In the EAF process, charged material (consisting of scrap metal and limes) is heated by means of an electric arc (Figure 10.4). The use of 100 percent scrap metal in the EAF process reduces the resources and energy required to produce steel. Steel scrap and lime are poured into the furnace, and the electrical current is passed through the electrodes, producing an electric arc on the scrap metal. The scrap is melted with the heat generated from the arc. Other metals, if necessary, are added at this stage to form special steels with required chemical compositions. Oxygen is blown into the furnace to remove impurities, such as aluminium, silicon, manganese, phosphorus, and carbon.

Figure 10.4 Electric arc furnaces



(Scanrail1/Shutterstock.com)

The metallic oxides thus formed are removed as slag. The oxidation reaction is exothermic, which further aids the melting of the scrap. Samples are taken to check the chemical composition of the steel for quality assessment. After removing the slag by tilting the furnace, the molten steel is removed. The EAF process is used for making special quality steels, such as stainless steel, by alloying iron with other metals. Iron is alloyed with chromium, nickel, and carbon to produce stainless steel that is resistant to corrosion (Ghosh and Chatterjee, 2008).

10.5 Wrought Iron

Wrought iron is a relatively pure iron alloy with very low carbon content. When iron ore is heated with charcoal or coal, the oxygen released from iron ore combines with carbon to form carbon monoxide and carbon dioxide. It produces a spongy, porous mass of relatively pure iron (called bloom) that contains traces of charcoal and slag. Most of the impurities and slag are driven off by hammering, while hot, to produce wrought iron (which means "worked" iron). The carbon content in wrought iron is very low (0.02–0.08%, absorbed from charcoal) and contains traces of slag (1–2%), which gives its fibrous look. Wrought iron also contains some manganese (<0.1%) and silicon (0.2%).

Wrought iron has been produced since the days of Iron Age, but its use in construction industry began in the 19th century because of its strength in tension, ductility, malleability, and weldability. Tensile strength of wrought iron along the grain ranges from 310–380 MPa (45–55.1 ksi); tensile strength is lower across the grain. The world-famous Eiffel Tower (Figure 10.5) in Paris was built in 1889, using wrought iron.

Eiffel Tower, Paris France

Figure 10.5

(Courtesy of N. Sivakugan, James Cook University, Australia)

With the production of steel with better structural performance, the use of wrought iron is now mostly limited to decorative purpose only.

10.6 Cast Iron

Cast iron is alloy primarily composed of iron, carbon, and silicon. It has relatively high carbon content (2–4%), compared to wrought iron and steel. Iron, when heated at high temperature, absorbs carbon rapidly, thus reducing its melting point and resulting in molten iron. Flux, such as limestone, helps remove undesirable impurities. Cast iron is often produced by remelting pig iron and scrap iron, along with limestone and coke (fuel made by baking coal). Carbon and silicon content are adjusted to the range between 2.1–4 percent and 1–3 percent, respectively. The molten iron is then cast into desired shapes. Grey cast iron is the most commonly produced cast iron and contains free carbon flakes distributed throughout the metal. Grey cast iron is easy to cast but cannot be worked. Another type of cast iron is white cast iron where carbon content is combined chemically as carbide of iron. White cast iron has better tensile strength and malleability compared to grey cast iron.

Due to its high carbon content, cast iron is hard, brittle, and not malleable. In the late 18th century, cast iron was used in the construction of bridges and buildings. Cast iron is strong in compression but weak in tension; hence arches which create compression forces in the member are the suitable structure type for cast-iron construction. Columns in buildings, which carry compression load, were also built using cast iron. However, due to its low tensile strength and brittle nature, cast iron was replaced by steel by the end of the 19th century. Steel has much better structural performance, due to its ductility and high tensile strength, compared to cast iron. Cast iron is, however, still used in making cookware (Figure 10.6), piping, and engine blocks.

10.7 Carbon Steel

As discussed in earlier sections, pure iron with a very little carbon content is ductile and malleable but soft, whereas cast iron with high carbon content is tough but has low tensile strength and is brittle. Carbon steel, on the other hand, is an alloy of iron containing carbon in the range 0.12–2 percent, manganese (typically limited to

Figure 10.6 Barbeque grill made of cast iron

(Courtesy of R. Tuladhar, James Cook University, Australia)

1.65%) and silicon (limited to 0.6%). Manganese improves strength and ductility of steel, whereas silicon improves strength. Based on the carbon content in steel, it can be classified as low-carbon steel, medium-carbon steel, and high-carbon steel. Change in carbon content in steel changes its properties like strength, ductility, and toughness.

Steel with carbon content in the range of (less than 0.25%) is classified as low-carbon steel. Low-carbon steel is also known as mild steel (Figure 10.7a), and it is the most commonly used steel. It is used for automobile chassis, cookware, pipelines, and the construction industry. Mild steel has good ductility, weldability, strength, and is the least expensive. It, however, has poor corrosion resistance, and it must be painted or protected against corrosion. Steel containing 0.3–0.6% carbon is classified as medium-carbon steel. Most of the structural steel used in construction industry fall under the low-carbon and medium-carbon categories.

An increase in carbon content in steel increases its tensile strength and yield strength (Figure 10.8) and adversely affects its ductility (Figure 10.9), weldability,

(a) Mild steel (b) Prestressing tendons used in prestressed concrete **Figure 10.7**


(a)
(b)

(Courtesy of R. Tuladhar, James Cook University, Australia)

Effect of carbon content on the tensile and yield strength of carbon steel **Figure 10.8**

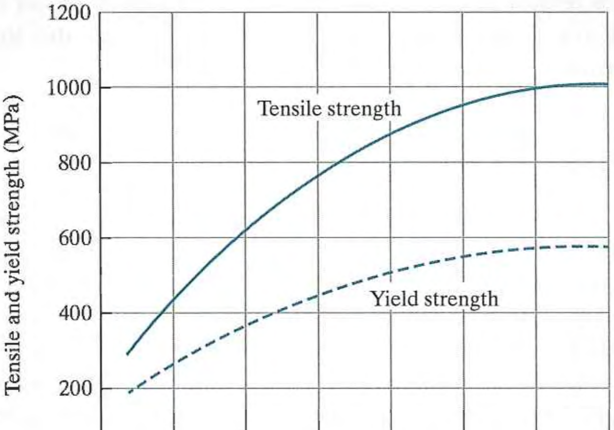
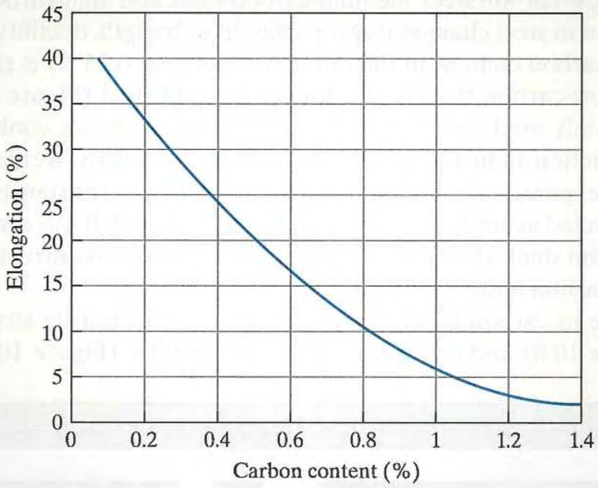


Figure 10.9 Effect of carbon content on the elongation of carbon steel



and impact toughness. Steel containing carbon in the range of 0.6–0.95 percent and manganese in the range of 0.3–0.9 percent is classified as high-carbon steel. High-carbon steel is very strong and is used for making springs, high tensile strength prestressing wires (Figure 10.7b) for prestressed concrete, and high strength bars. Steel with 0.96–2 percent carbon is classified as very high-carbon steel and is extremely strong but is very brittle.

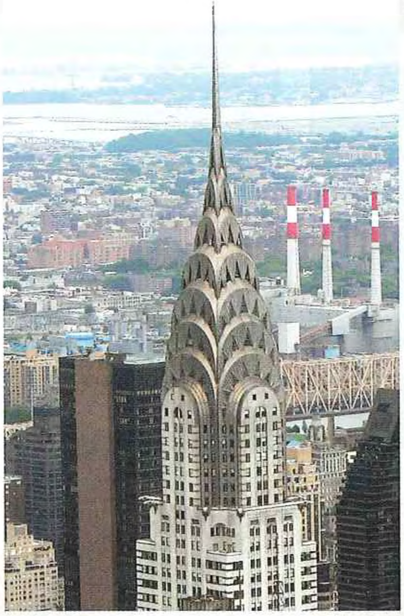
Steel containing other elements, such as nickel or chromium, is called alloy steel and has specific applications. Steel alloy with at least 11 percent chromium is called stainless steel. Chromium in stainless steel reacts with oxygen in the atmosphere to form a thin layer of chromium oxide on the surface. This thin passive layer formed on the surface of stainless steel makes it highly resistant to corrosion and stains. Stainless steel is used in construction industry mainly for aesthetic reasons and for special structures, which require high resistance to corrosion. A very good example of the use of stainless steel is the stainless steel clad crown of Chrysler Building (Figure 10.10) in New York.

10.8 Structural Steel

Structural steel used in construction industry is mainly low-carbon or medium-carbon steel. As described in Section 10.4, structural steel is predominantly produced, using the basic oxygen steelmaking (BOS) and the electric arc furnace (EAF) processes. Molten steel from BOS and EAF is either poured through ladles to cast ingots (metal blocks used for further processing) or is continuously casted (Figure 10.11) into solidified semifinished products like billets, blooms, or slabs, depending on the end-use of the product and are subsequently processed in finishing mills. Billets are lengths of metal with a square or round cross section with an area less than 230 cm^2. Billets are hot-rolled into bars and wires (Figure 10.12a). Blooms (Figure 10.12b) have cross-sectional areas larger than billets and go through hot-rolling process to produce

Crown of the Chrysler Building made of stainless steel

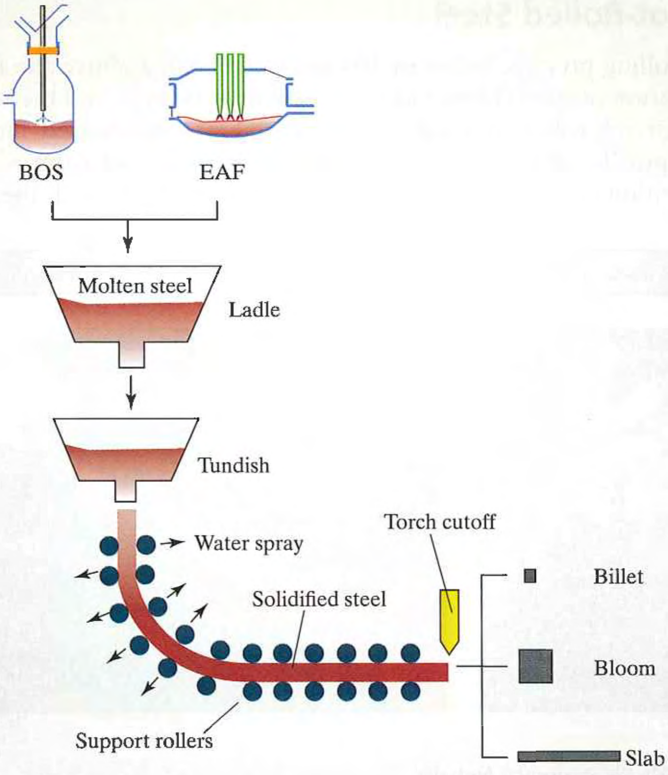
Figure 10.10



(Courtesy of R. Tuladhar, James Cook University, Australia)

Continuous casting of steel

Figure 10.11



(Courtesy of R. Tuladhar, James Cook University, Australia)

Figure 10.12 (a) Steel billets (b) Steel blooms


(a) (b)

(Courtesy of S. Kelly, OneSteel Whyalla Steelworks, Australia)

different structural shapes (Figure 10.13). Slabs on the other hand, are flat and wide rectangular metal sheets and are processed into sheet metal, plates, pipes, and tubes. Structural steel with required shapes is then produced through hot-rolling and cold-formed processes. Continuous casting together with the hot-rolling process is predominantly used in modern steel plants due to its better productivity and yield and lower energy consumption.

10.8.1 Hot-Rolled Steel

In the hot-rolling process, billets or blooms are reheated above the recrystallization temperature of steel (1200–1300 °C) in a reheat furnace, and the red hot metal is pushed through rollers that squeeze the metal into the desired shape, depending on the profile of the rollers (Figure 10.13). Hot steel from a continuous-casting operation can also be rolled by directly passing it through the rolling mills

Figure 10.13 (a) Channel section going through a rolling mill (b) Rolling mill


(a) (b)

(Courtesy of S. Kelly, OneSteel Whyalla Steelworks, Australia) without the need to reheat. Structural steel members with various sectional shapes (Figure 10.14), such as wide-flange sections, I-beams, channel sections, equal or unequal angles, sheet piling, rails, and bars, are produced by the hot-rolling process. Dimensions of standard steel sections and their sectional and mechanical properties are listed in design standards, such as ASTM A6 (2014). These structural steel products are used in buildings as columns, beams, and bracing members; as members of trusses; and as bridge girders.

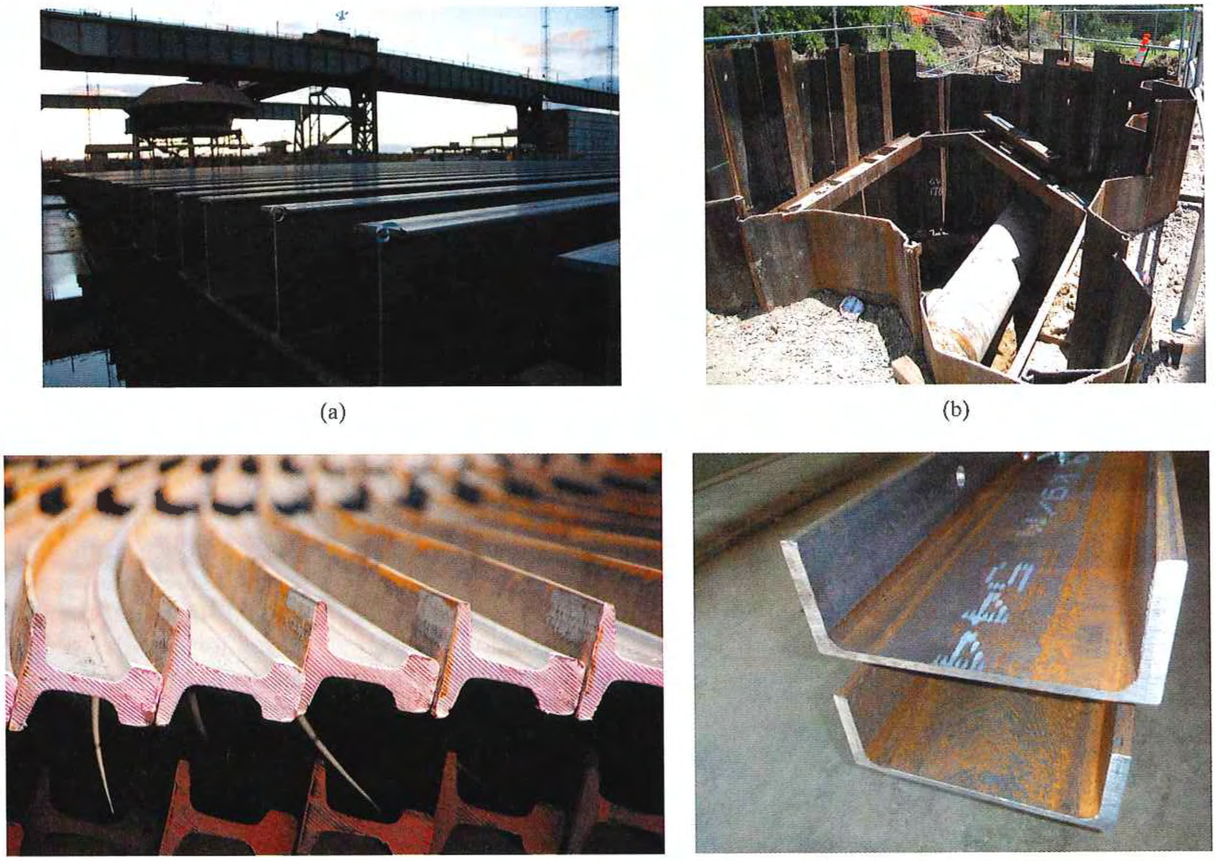
Steel produced through the hot-rolling process is relatively cheaper than cold-formed steel. Hot rolling, along with a suitable heat treatment process, enhances mechanical properties of the steel as well.

Structural Steel Grades

The American Society for Testing and Materials (ASTM) Standards A36, A529, A572, A242, A588, and A514 specify chemical and mechanical properties for hot-rolled structural steel. A36 steel with carbon content of 0.25–0.29 percent is the most

Hot-rolled sections: (a) I sections in cooling yard (b) Sheet piles (c) Rail sections (d) Channel sections

Figure 10.14



(a) (b)

(c) (d)

((a and c) Courtesy of S. Kelly, OneSteel Whyalla Steelworks, Australia)
((b) Courtesy of N. Sivakugan, James Cook University, Australia)
((d) Courtesy of R. Tuladhar, James Cook University, Australia)

commonly used structural steel in building and bridge construction (ASTM A36, 2012). It has a minimum yield stress of 250 MPa (36 ksi) and ultimate tensile strength in the range of 400–550 MPa (58–80 ksi). ASTM A529 covers high-strength carbon-manganese steel of yield stress 340 MPa (50 ksi) and 380 MPa (55 ksi). A529 steel has maximum carbon content of 0.27 percent and maximum manganese content of 1.35 percent (ASTM A529, 2014).

High-strength, low-alloy (HSLA) steels or high-strength steels contain other alloying elements (such as chromium, columbium, copper, manganese, molybdenum, nickel, vanadium, and zirconium) in addition to carbon. These additional alloying elements improve mechanical properties and weldability of the steel. The term low-alloy in HSLA means that the total alloying elements is less than 5 percent of the total composition of steel. ASTM A572 (2013) is high-strength, low-alloy columbium-vanadium structural steel and is produced in Grades 42, 50, 55, 60, 65 representing yield stresses of 42 ksi (290 MPa), 50 ksi (345 MPa), 55 ksi (380 MPa), 60 ksi (415 MPa), and 65 ksi (450 MPa). ASTM A242 (2013) and A588 HSLA steels have nickel and copper as the principal alloying elements, which enhance their atmospheric corrosion resistance and are primarily used as structural members where added durability are important (ASTM A588, 2010). In these corrosion-resistant HSLA steels, a dense layer of oxide film (called a patina) is formed which seals and protects the base steel from further corrosion.

Quenched and tempered alloy steels (ASTM A514, 2014) have a higher percentage of alloying elements than HSLA and are heat treated by quenching and tempering (Section 10.9) to improve their strength and other mechanical properties. These steels exhibit high-yield stress in the range of 90–110 ksi (620–760 MPa); however, the ductility for these steels is significantly lower than carbon and HSLA steels. Low carbon content and the quenching process enhance the strength in these steels, and the tempering process improves the ductility. Stress-strain curves for these different types of steel are shown in Figure 10.21 in Section 10.10.

10.8.2 Cold-Formed Steel

Cold-formed steel is produced by rolling or pressing thin steel sheets through rollers or press brakes at room temperature to produce lightweight steel sections. Steel sheets used to produce cold-formed steel sections are generally 0.378 mm (0.0149 in.) to about 6.35 mm (0.25 in.). Cold-formed steel sections are usually used for lightweight and short-span structures, which require better durability and dimension precision (Yu and LaBoube, 2010). More precise dimension sections can be achieved in cold-formed sections compared to hot-rolled steel products as the hot-rolled steel members tend to shrink after cooling. Cold-formed steels are used as structural framing members or in panels and decks. Individual structural framing members can be made of channel section (C-section), Z-sections, angles, Sigma sections, I-sections, T-sections, and tubular members. Cold-formed steel sections are usually used as primary load-bearing systems for low-rise, light structures, space frames, or in storage racks (Figure 10.15). They are also used as joists, purlins, studs, and decks in high-rise structures, along with hot-rolled sections as a main framing system (Figure 10.16). Cold-formed sections are also widely used in roof and floor decks and wall panels, which carry load as well as provide a surface to attach flooring and roofing materials.

ASTM A1003 (2013) provides specifications for steel sheet, carbon, metallic- and nonmetallic-coated cold-formed sections for framing members.

(a) Cold-formed sections used for storage racks (b) Cold-formed steel used in floor decks and storage racks

Figure 10.15

(a)

(b)

((a) Courtesy of R. Tuladhar, James Cook University, Australia)
((b) Courtesy of D. Stephenson, Rockfield Technologies, Australia)

Building frame with hot-rolled steel used as columns and main beams and cold-formed steel used as purlins, girts, joists, and roof/wall cladding

Figure 10.16

(Courtesy of D. Stephenson, Rockfield Technologies, Australia)

10.8.3 Reinforcing Bars

Concrete, as described in Chapter 8, is the most commonly used construction material. Unlike steel that has similar strength in both compression and tension, concrete is strong in compression while very weak in tension. Hence, concrete structures need steel reinforcement to take the tensile stresses. Steel reinforcement in concrete structures increases its ductility and improves its resistance to cracking (Figure 10.17).

Figure 10.17 Steel reinforcement in concrete bridge pier

(Courtesy of R. Tuladhar, James Cook University, Australia)

Steel can be used in concrete as a normal reinforcement or as prestressed reinforcement. The main difference between the conventional reinforced concrete structures and prestressed concrete structures is that the steel members in reinforced concrete structures are not stressed, whereas the prestressed strands or bars in prestressed concrete structures are in tension, and they will exert prestressing force on the concrete structures which counteracts the load acting on the structure.

For conventional reinforced concrete structures, reinforcing steel can be plain bars or deformed bars (Figure 10.18a) or in the form of plain and deformed wire mesh (Figure 10.18b). Plain bars are smooth reinforcing bars without any deformation on the surface. Plain bars have limited bonding within the concrete matrix. The deformation present on deformed bars helps develop better bonding between concrete matrix and steel. Steel used to reinforce concrete structures consists mostly hot-drawn deformed steel bars. ASTM A615 (2014) gives standard specifications for deformed and plain carbon-steel bars for concrete reinforcement. Plain and deformed wire mesh is a prefabricated rectangular grid of steel wires (can be plain or deformed and available in rolls or sheets) which is mainly used to reinforce concrete roof and floor slabs and concrete footpaths. Steel mesh in slabs resists tensile stresses as well as shrinkage and temperature stresses.

In prestressed concrete structures, prestressing steel is stressed either before or after the concrete is being cast. If the steel is stressed before the concrete is cast, it is called pretensioning; whereas if the steel is stressed after concrete gains its strength, it is called posttensioning. In either case, steel in prestressed concrete is always in tension. Prestressing steel will exert compressive force on the concrete member, which counteracts with the self-weight and external load acting on the concrete member. Because prestressing steel is stressed to high stresses, the members are made of high-carbon high-strength wires, strands, cables, and bars (Figure 10.7b). ASTM A416 (2005) and ASTM A421 (2010) cover the standard specification for steel strand, wire, and stress-relieved steel wire for prestressed concrete. Prestressed concrete is usually used for long-span bridges, high-rise buildings, and precast elements.

(a) Plain and deformed steel bars (b) deformed steel mesh Figure 10.18

(a) (b)

((a) Courtesy of S. Moraga)
((b) Courtesy of R. Tuladhar, James Cook University, Australia)

10.9 Heat Treatment of Steel

Steel can be subjected to heat treatment to achieve different physical and mechanical properties. The process involves heating and subsequently cooling the steel in solid state, which affects its properties due to the changes in its crystalline structure (grain size). Heat treatment is usually done in high-carbon steel to enhance steel properties like strength, machinability, formability, and ductility. The heat treatment is carried out after the steel sections are given their final form (shape) in which they will be used. It commonly includes following three processes.

10.9.1 Quenching

This process includes heating steel above the transformation range (about 910 °C or 1670 °F for pure iron when crystal structures in iron change from ferrite to austenite) and rapidly cooling the hot steel using water or oil (Vinnakota, 2006). The surface of the steel that comes in contact with the coolant cools off rapidly, resulting in the formation of a fine-grained structure known as martensite steel. This results in very strong and hard steel surface but one susceptible to cracking. The rate of cooling in the interior of thick steel sections is much slower and not enough to induce the hardening of the steel.

10.9.2 Tempering

The quench-hardened steel is subjected to tempering to improve ductility. In this process, quenched steel is reheated to a temperature lower than critical temperature (temperature at which austenite starts to form) and is allowed to cool slowly in air. The slow cooling of steel relieves the internal stresses and decreases the hardness, while increasing the toughness and ductility of steel.

10.9.3 Annealing

Annealing includes heating the steel above its transformation range and maintaining the temperature above transformation range for sufficient time, followed by slow cooling to room temperature. Annealing improves ductility, removes internal stresses, and softens the steel, making it suitable for further work such as shaping, forming, and stamping.

10.10 Mechanical Properties of Steel

For structural steel, mechanical properties, such as stress-strain curve, tensile strength, yield stress, stiffness, and ductility, are the important parameters for design. Mechanical testing is conducted on steel members or specimens extracted from steel sections to obtain these properties. ASTM A370 (2014) covers the standard test methods and definitions of the various mechanical testing of steel products.

10.10.1 Tension Test

A tension test is used to determine the yield stress, ultimate tensile strength, stiffness, and ductility of steel and is carried out on a flat or dog-bone specimen (called a coupon) cut out of a steel section (Figure 10.19). The test coupon is gripped at both ends in the tensile testing apparatus and is pulled at a constant speed until it breaks. Gauge length (the length of the specimen between the grip over which elongation

Figure 10.19 Tensile test coupons before and after the test

(Courtesy of R. Tuladhar, James Cook University, Australia)

is measured) and the initial cross-sectional area of the specimen is measured before the test.

During the tensile test, the force applied and elongation of the specimen are recorded. Elongation of the gauge length is measured using an extensometer, which is used to calculate strain (change in the gauge length over the original gauge length, L_0 Equation 10.2) in the member. Stress in the specimen during the test is measured as the load applied divided by the original cross-sectional area (A_0) of the specimen (Equation 10.1). The results of the tensile test are presented as a stress-strain curve (Figure 10.20)

$$f = {}^{T}\!/_{A_0} \tag{10.1}$$

$$\varepsilon = {}^{e}\!/_{L_0} = \frac{(L - L_0)}{L_0} \tag{10.2}$$

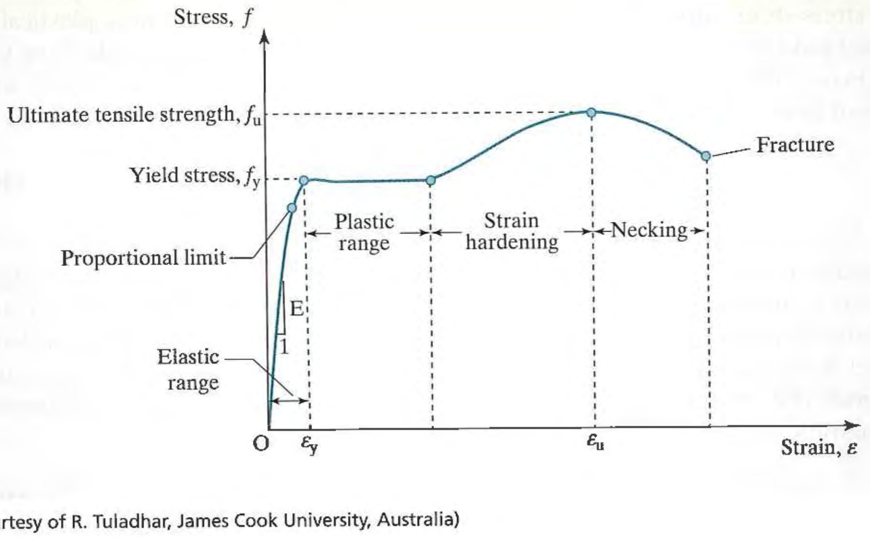
where f and ε are stress and strain, respectively, and L is the new length of the member when tensile load T is applied.

Idealized stress-strain curve for steel is shown in Figure 10.21. As seen in the stress-strain curve, steel initially behaves in a linear elastic manner. In this linear elastic range, the material follows Hooke's Law, which means that the stress is directly proportional to the strain applied. The slope of the stress-strain curve in the linear elastic range is called Young's modulus or the elastic modulus (E), which represents the rigidity or stiffness of the material (Equation 10.3). It is measured as a ratio of the stress (f) to the corresponding strain (ε) within the elastic range. Young's modulus of steel (E) varies between 200,000 MPa to 210,000 MPa, and E is generally taken as 200,000 MPa in steel design. Within the elastic range,

$$E = {}^{f}\!/_{\varepsilon} \tag{10.3}$$

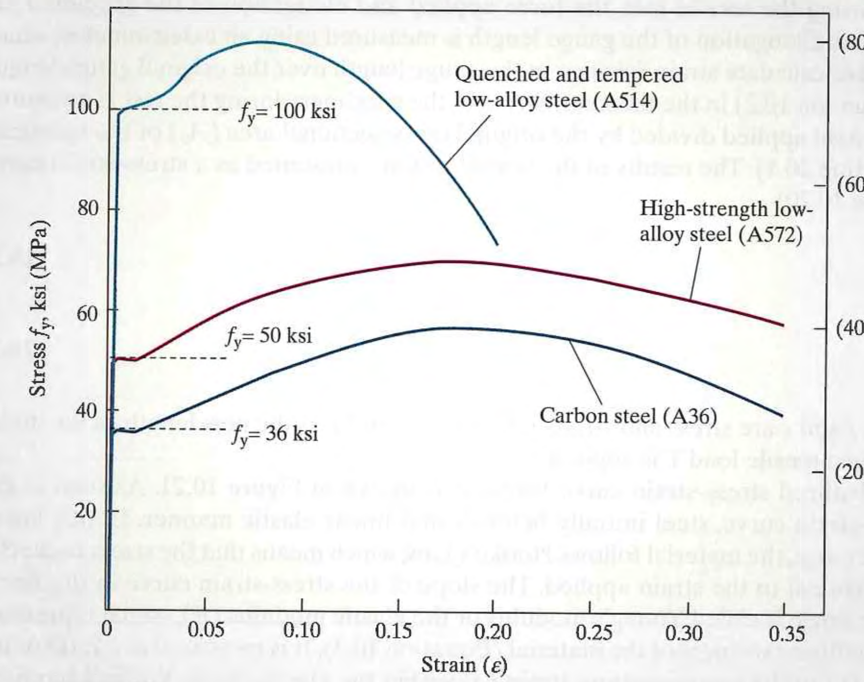
Idealized stress-strain curve for structural steel (not to scale)

Figure 10.20



(Courtesy of R. Tuladhar, James Cook University, Australia)

Figure 10.21 Typical stress-strain curves for different types of structural steel



(Based on Harris, Harry G. and Sabnis, Gajanan (1999). Structural modelling and experimental techniques. Boca Raton, CRC Press LLC.)

Within the elastic range, the steel remains elastic and recovers perfectly on unloading. The stress-strain curve is linear up to the point called the proportional limit. At a slightly higher load than the proportional limit, the steel starts to behave plastically which means strain in the steel increases considerably without any increase in the stress. This plastic range is represented by the plateau (known as yield plateau) in the stress-strain curve. The stress at which steel starts to yield or behave plastically is called yield stress (f_y). The corresponding strain at the yield stress is called the yield strain (ε_y). The yield stress (f_y) is a very important property of structural steel, and it is used in steel design (Equation 10.4).

$$f_y = E \times \varepsilon_y \tag{10.4}$$

Beyond the yield plateau, stress starts to again increase beyond yield stress with an increase in the strain. This phenomenon is called strain-hardening. Plastic deformation in the steel during the yield plateau causes dislocation of the crystal structure, making the metal stronger. This effect is called strain-hardening. The strain-hardening effect decreases with the increase in strain. The maximum stress in the stress-strain diagram (Figure 10.20) is identified as the ultimate tensile strength (f_u) of the steel (Equation 10.5).

$$f_u = {}^{T_u}\!/_{A_0} \tag{10.5}$$

where T_u is the maximum tensile force achieved in the tension test.

The cross-sectional area of the steel is markedly reduced, and the specimen visibly stretches in length. This stage is called necking (Figure 10.19). The specimen finally fails at the fracture strain (ε_u).

Ductility (δ) of the material is its ability to undergo a large deformation without breaking (Equation 10.6). It can be represented as the percentage of elongation of the test specimen after it breaks.

$$\delta = \frac{L_f - L_0}{L_0} \tag{10.6}$$

where L_f is the final length of the specimen after breaking.

10.10.2 Poisson's Ratio (v)

Poisson's ratio (v) in structural steel can approximately be taken as 0.3 in the elastic range and 0.5 in the plastic range.

10.10.3 Shear Modulus of Elasticity (G)

Shear modulus of elasticity (G) of a material is defined as the ratio of shear stress to shear strain in the elastic range. Shear modulus is taken as 80,000 MPa for structural steel. The relation between Young's modulus (E) and shear modulus (G) is given by Equation 10.7.

$$G = \frac{E}{2(1 + v)} \tag{10.7}$$

where v is the Poisson's ratio of the material.

EXAMPLE 10.1

A tensile test is carried out on a steel rod of 12 mm diameter and gauge length of 100 mm. The yield strength of the steel (f_y) is 500 MPa. At the tension load of 45 kN, calculate stress and strain in the steel rod, taking Young's modulus (E) as 200,000 MPa. Find out if the steel rod is in elastic region or has already yielded.

Solution:

Original cross-sectional area of the steel bar $(A_0) = \frac{\pi}{4}d^2 = \frac{\pi}{4}12^2 = 113.1 \text{ mm}^2$

Tension load acting on the bar $(T) = 45$ kN

Stress in the steel at load 45 kN $= f_y = {}^{T}\!/_{A_0} = 45 \times {}^{1000}\!/_{113.1} = 397.9 \; {}^{\text{N}}\!/_{\text{mm}^2}$

Strain in the steel at load 45 kN $= \varepsilon = {}^{f_y}\!/_{E} = {}^{397.9}\!/_{200,000} = .00198 \; {}^{\text{mm}}\!/_{\text{mm}}$

As the stress in the steel bar at 45 kN load is 397.9 MPa, which is less than the yield stress of 500 MPa, the steel bar is in the elastic range.

EXAMPLE 10.2

A bar made of carbon-steel of size 8 mm × 12 mm has a yield stress of 250 MPa and ultimate tensile strength of 440 MPa. The gauge length of the specimen tested under tension loading is 100 mm.

a. Determine its elongation when a tension load of 20 kN is applied.

b. Determine the maximum load the bar can take before it fractures.

Take Young's modulus of steel as 200,000 MPa.

Solution:

Original cross-sectional area of the steel bar $(A_0) = 8 \times 12 = 96 \text{ mm}^2$

a. Stress in the steel bar at load 20 $kN = f_y = {}^{T}\!/_{A_0} = {}^{20 \times 1000}\!/_{96} = 208.3 \; {}^{\text{N}}\!/_{\text{mm}^2}$

Strain in the steel bar at load 20 $kN = \varepsilon = {}^{f_y}\!/_{E} = {}^{208.3}\!/_{200{,}000} = .00104 \; {}^{\text{mm}}\!/_{\text{mm}}$

Elongation in the steel bar at load 20 $kN = \Delta L = \varepsilon \times L_0 = 0.00104 \times 100 = 0.104$ mm

b. Maximum load the bar can take $= T = f_u \times A_0 = 440 \times 96 = 42240 \text{ N} = 42.24 \text{ kN}$

10.11 Summary

1. Structural steel used in construction industry is an iron alloy containing carbon in the range of 0.2–1.5% by weight and other elements.
2. The basic oxygen steelmaking (BOS) and electric arc furnace (EAF) processes together account for 98 percent of the total steel produced in the world today.
3. In the BOS process, pig iron and scrap iron are converted into steel by blowing pure oxygen over the molten iron from blast furnace. Lime-based fluxes are used to remove impurities in the form of slag.
4. In the EAF process, scrap iron and lime are heated by means of an electric arc. Oxygen is blown into the furnace to remove other impurities from the iron.
5. Most of the structural steel used in the construction industry falls under low-carbon (carbon content less than 0.25%) and medium-carbon steel (carbon content in the range of 0.3 to 0.6%) category. The increase in carbon content in steel increases its strength and toughness but adversely affects its ductility.
6. Continuous casting, together with the hot-rolling process, is predominantly used in modern steel plants to produce desired steel sections for the construction industry. In the hot-rolling process, red-hot iron is squeezed through rollers into desired shapes.
7. Cold-formed steel is produced by rolling or pressing thin steel sheets through rollers or press brakes at room temperature to produce lightweight steel sections.
8. Hot-rolled sections are subjected to heat treatment, which involves heating and subsequent cooling of steel to enhance physical and mechanical properties of steel.

9. The quenching process, which involves rapid heating of steel followed rapidly cooling, results in a very strong and hard steel surface. Ductility is enhanced when quench-hardened steel is reheated and allowed to cool slowly. This process is called tempering.
10. A tension test is used to determine the yield stress, ultimate tensile strength, stiffness, and ductility of steel. The stress at which steel starts to yield is called yield stress. The corresponding strain at the yield stress is called yield strain.

Exercises

1. Which one of the following statements is false?
   - **a.** Steel is 100% recyclable.
   - **b.** Steel members have a high strength-to-weight ratio.
   - **c.** Strength in steel is enhanced at high temperature caused by fire.
   - **d.** Prevention measures should be taken to prevent the corrosion of steel.
2. Wrought iron has carbon content in the range of:
   - **a.** 0.02–0.08%
   - **b.** 0.2–0.3%
   - **c.** 0.6–0.95%
   - **d.** 2–4%
3. The process of melting metal out of its ore in the presence of reducing agents to remove impurities is called:
   - **a.** Continuous casting
   - **b.** Hot rolling
   - **c.** Smelting
   - **d.** Casting
4. High carbon content in cast iron makes it
   - **a.** Hard and brittle
   - **b.** Soft and ductile
   - **c.** Malleable
   - **d.** Hard and ductile
5. The chief alloying element that gives corrosion-resistant properties to stainless steel is:
   - **a.** Carbon
   - **b.** Phosphorous
   - **c.** Silicon
   - **d.** Chromium
6. The process of reheating steel above recrystallization temperature and pushing the red-hot metal through rollers to squeeze it into the desired shape is called:
   - **a.** Hot-rolling process
   - **b.** Cold-forming process
   - **c.** Quenching
   - **d.** Tempering

7. The tempering process enhances:
   - a. Strength of steel
   - b. Ductility of steel
   - c. Carbon content of steel
   - d. Corrosion resistance of steel

8. Hooke's Law is valid within the:
   - a. Plastic range
   - b. Strain-hardening range
   - c. Elastic range
   - d. Necking stage

9. The ability of material to undergo large tensile deformation without breaking is called:
   - a. Strain hardening
   - b. Ductility
   - c. Weldability
   - d. Malleability

10. Prestressing steel used in prestressed concrete structures is made of
    - a. Mild steel
    - b. High-carbon, high-strength steel
    - c. Cast iron
    - d. Wrought iron

11. Give five reasons why steel is widely used as a construction material for infrastructure and the building industry.

12. What are the limitations of structural steel, and what are the preventive measures taken to overcome these limitations?

13. Explain the basic oxygen steelmaking and electric arc furnace processes, highlighting their advantages over previously used steel-making processes.

14. Describe the main differences between wrought iron, cast iron, and structural steel.

15. Why do we need to use steel reinforcement in reinforced concrete structures?

16. Explain what the ductility of a material means. How do carbon content and heat treatment affect the ductility of steel?

17. As a consulting engineer, you are asked to suggest preventive measures to prevent corrosion in steel girders of a bridge in a marine environment. Explain various methods available for corrosion protection of steel structures.

18. Describe the production process of hot-rolled steel and cold-formed steel. Give examples where these two types of structural steel are used.

19. Draw a typical stress-strain curve for steel subjected to tension and identify various important parameters in the curve

20. A sample of a 20 mm reinforcing steel used in concrete structure is subjected to a tensile test. The gauge length of the test specimen is 150 mm. From the stress-strain

curve, it was identified that the yield stress of the steel was 550 MPa. Determine the corresponding yield strain. Take Young's modulus as 200,000 MPa.The steel bar fractured at 210 kN. Find out the ultimate tensile strength of the steel.

21. A bar, made of high-strength low-carbon steel of Grade 50 (yield strength grade 50 ksi), is 0.5 in. thick and 1.2 in. wide. The ultimate tensile strength of the steel is 70 ksi. Determine the load at which the bar starts to yield and also determine the load when the bar is likely to fracture.

Taking the original gauge length as 5 in. and Young's modulus of steel as 30×10^6 *psi*, estimate the new gauge length when the specimen is stressed at 2/3 of the yield stress.

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CHAPTER 11

The bubble-like exterior cladding of the Water Cube in Beijing is made of ETFE (ethylene tetrafluroethylene), a tough durable plastic. robinimages2013/Shutterstock.com

Polymers, Ceramics, and Composites

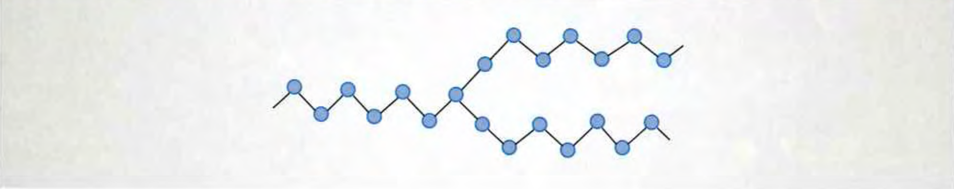
11.1 Introduction

Nonmetallic materials such as polymers, ceramics, and composites have been gaining high interest over the past few decades due to their attractive physical and mechanical properties. Polymers are long-chain molecules that are naturally present and can also be chemically synthesized. They are lightweight, highly ductile, and easily fabricated into different shapes and sizes. The low melting point of polymers restricts their use in high-temperature applications. Also, their mechanical strength is inferior as compared to metals. On the other hand, ceramics have high melting points and good mechanical strength. They are basically composed of metallic and nonmetallic elements. Ceramics have poor electrical and thermal conductivity and hence are used as insulating materials. They are also used in handling corrosive solutions due to their high resistance to corrosion. Composites are another class of materials, which combine two or more of the basic materials (metals, ceramics, and polymers). They have excellent combination of properties such as low density, high strength, and flexibility. Composites have widespread applications in aerospace and automobile industries, and also in high-end sports equipment. In this chapter, the physical, chemical, and mechanical properties of polymers, ceramics, and composites are discussed.

11.2 Polymers

Polymers are macromolecules containing many repeated basic units or monomers (Figure 11.1). They are synthesized from monomers, and the process is commonly known as polymerization. The polymerization process can be divided into two types: (1) step-growth or condensation polymerization, and (2) chain-growth or addition polymerization. In step-growth polymerization, small chains of monomers combine to form large chains, which is a stepwise intermolecular chemical reaction (for example,

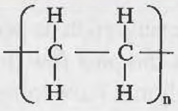
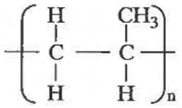
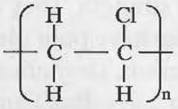
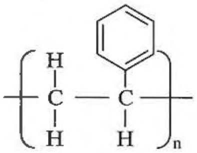
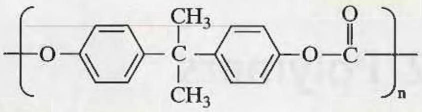
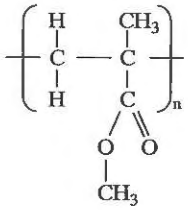
Figure 11.1 An example of monomer arrangement in a polymer chain

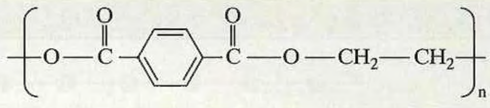
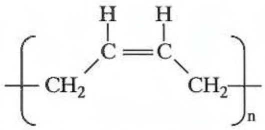
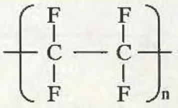
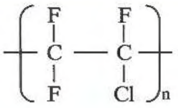


polyester, polycarbonates, and polyamides), whereas in chain-growth polymerization, the growth of the polymer chain is only by individual monomer addition (for example, polyethylene and poly(methyl methacrylate)).

Polymers can be broadly classified into two types: natural polymers (such as natural rubber, wool, and cellulose) and synthetic polymers (such as polyethylene, polystyrene, polypropylene, and polyvinyl chloride). Table 11.1 shows a list of well-known commercial polymers and their structures. The microstructure of polymers strongly influences their properties. Due to their large molecular mass and chain-like structure, the polymers possess unique physical and mechanical properties.

Table 11.1 Commercial polymers and their chemical structures

Polymer	Chemical Structure
Polyethylene	 $\left(-\overset{\displaystyle H}{\underset{\displaystyle H}{C}}-\overset{\displaystyle H}{\underset{\displaystyle H}{C}}-\right)_n$
Polypropylene	 $\left(-\overset{\displaystyle H}{\underset{\displaystyle H}{C}}-\overset{\displaystyle CH_3}{\underset{\displaystyle H}{C}}-\right)_n$
Polyvinyl chloride	 $\left(-\overset{\displaystyle H}{\underset{\displaystyle H}{C}}-\overset{\displaystyle Cl}{\underset{\displaystyle H}{C}}-\right)_n$
Polystyrene	
Polycarbonate	
Poly(methyl methacrylate)	 $\left(-\overset{\displaystyle H}{\underset{\displaystyle H}{C}}-\overset{\displaystyle CH_3}{\underset{\displaystyle C(=O)OCH_3}{C}}-\right)_n$

Polyethylene terephthalate	
Polybutadiene	
Polytetrafluoroethylene	 $\left[\mathrm{CF_2}-\mathrm{CF_2}\right]_n$
Polychlorotrifluoroethylene	 $\left[\mathrm{CF_2}-\mathrm{CFCl}\right]_n$

There are various characterization techniques available to understand the physical and chemical properties of polymers. The crystalline structure of a polymer can be determined using X-ray techniques. Fourier transform infrared spectroscopy (FTIR) and Raman spectroscopy are used to find the chemical composition of a polymer. The average molecular weight of a polymer can be calculated using gel permeation chromatography. Differential scanning calorimetry (DSC) is used to determine the melting point and glass transition temperature of polymers.

The length of the polymer chain is commonly expressed by the degree of polymerization (the number of polymers in the chain) or by molecular weight. The higher the length of the polymer, the higher the degree of polymerization and also the molecular weight is. The chain length of the polymer strongly influences its physical and mechanical properties. An increase in chain length increases viscosity, melting and boiling points, glass transition temperature, strength, and toughness. Van der Waals attraction also increases with an increase in chain length, which consequently makes the polymer stronger.

11.2.1 Chemistry

Based on the arrangement and type of monomers, there are different types of polymers (Figure 11.2). When polymers contain single type of repeated unit, they are called *homopolymers* (for example, polystyrene), and when they contain mixed repeated units, they are named *copolymers* (for example, ethylene-vinyl acetate). In alternating copolymers, the monomers are arranged in alternative fashion, whereas in periodic copolymers, the monomers are arranged in a repeating sequence. When blocks of two or more homopolymers are bonded covalently, then it is called *block copolymers*.

Polymers are not always simple, long-linear-chain macromolecules, but they also can form branches or side chains. The presence of side chains with different units makes *graft copolymers*. There are various types of branched polymers, for example, dendrimers, brush polymers, star polymers, and comb polymers (Figure 11.3). The difference in the structures influences their physical properties, such as viscosity. *Tacticity* is described as the relative stereochemistry of chiral centers in neighboring

Figure 11.2 Types of polymers based on arrangement and types of monomers

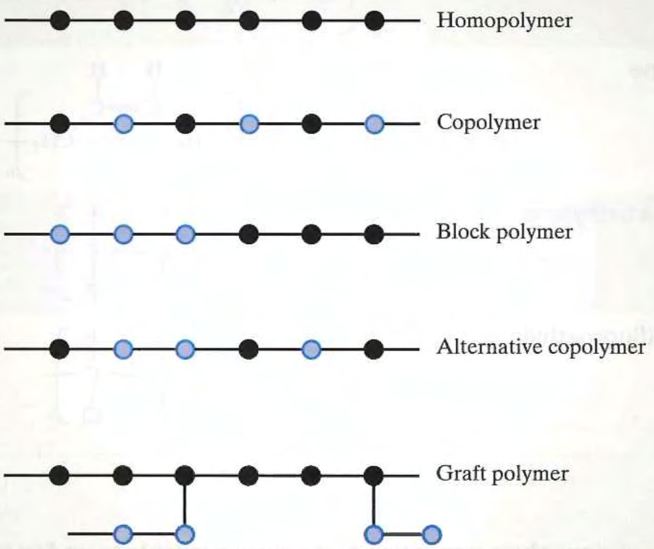
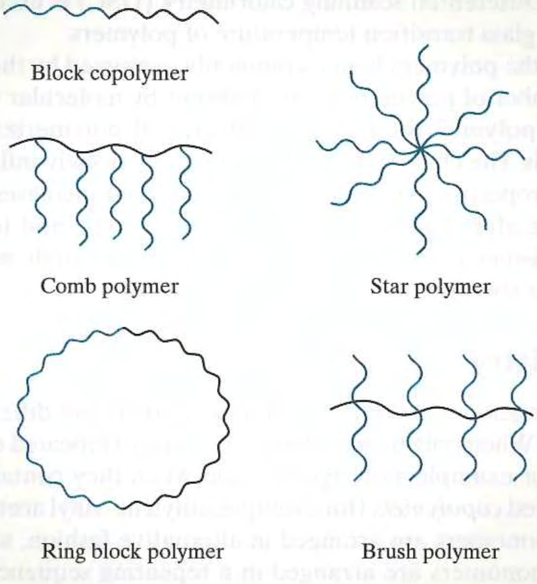


Figure 11.3 Types of branched polymers



structural units in a polymer. Tacticity can be of three types: (1) isotactic, where all substituents on the same side; (2) atactic, where substituents are randomly placed; and (3) syndiotactic, where substituents are alternatively placed.

Polyethylene is one of the widely used commercial polymers. It can be classified into four types based on its density and molecular weight: (1) ultra-high molecular weight polyethylene, which is tough and chemical resistant, has applications in automobile parts and bulletproof vests; (2) high-density polyethylene, which is used in outdoor furniture, water drainage pipes, and plastic containers; (3) medium-density polyethylene is used in gas pipes and packing film; and (4) low-density polyethylene is used for making retail store bags and squeeze bottles.

Polymers can be crystalline and/or amorphous. The properties of polymers are strongly influenced by the degree of crystallinity. For example, a polymer becomes transparent when the degree of crystallinity approaches zero. Crystalline melting point (T_m) is a transition temperature where a crystalline (or semicrystalline) phase is transformed to an amorphous phase. Polymer crystallization temperature influences the melting temperature of the polymer. Crystalline polymers have a sharp change in the density at their melting points. However, with semicrystalline polymers, the melting behavior is between that of amorphous and crystalline polymers.

Melting polymers involves breaking of the interchain bonds, and hence strong-chain polymers have high melting temperatures. Glass transition temperature (T_g) is when an amorphous polymer undergoes transition from a viscous liquid to an amorphous solid upon cooling. The glass transition temperature depends on the cross-linking, side-branching size of molecules/groups of the polymer. Generally, the glass transition temperature (T_g) of a polymer lies between 0.5 and 0.8 of its melting temperature (T_m), that is, $T_g = 0.5–0.8T_m$ (kelvin). Amorphous polymers have a small change in the slope of density at their glass transition temperatures.

Plasticizers are small molecules generally added to polymers to improve flexibility, for example, clothing and cling film wraps. They reduce the interactions within polymer chains and hence increase the mobility of chain segments under mechanical stress. The glass transition temperature is also decreased by addition of plasticizers. The addition of plasticizers largely reduces the cracking of plastic automobile parts under cold temperatures. They are added to polymers such as polyvinyl chloride to make inflatable products, where flexibility is required. Another additive material is sulphur, which is added to rubber to make it harder, and the process is known as vulcanization. This process is an irreversible chemical reaction occurring at high temperatures. In this process, the sulphur atoms cross-link the polymer chains and improve the strength. Carbon is also added to synthetic rubber to increase the strength.

11.2.2 Thermosetting and Thermoplastic Polymers

Polymers can be broadly classified into two types: (1) thermosetting polymers, and (2) thermoplastic polymers. Thermosetting polymers, also known as thermosets, are produced by heating, chemical reaction, or irradiation of pre-polymer liquids. These polymers, once cured, become hard, and they cannot be remolded or melted. In other words, they do not soften when heated again, the chemical bonding is irreversible, and the hardness is permanent. In thermosetting polymers, cross-linking through active sites creates a three-dimensional network structure which increases their molecular weight and melting points. Further, the cross-link bonding increases the strength and dimension stability of the polymers. Hence, these thermosets have been used in high temperature applications. However, when these polymers are overheated, they will decompose without melting; hence they are not recyclable. Thermosets are molded into different shapes and sizes by a process called *curing*. Molding can be done using various techniques such as extrusion molding, compression molding, spin casting, and injection molding. Some examples and applications of thermosetting polymers: polyurethanes used in flooring, synthetic fibers, and mattresses; bakelite used in electric insulators; and polyimides used in printed circuit boards.

Thermoplastic polymers, also known as thermoplasts or *thermosoftening plastics*, are polymers which soften and form a viscous liquid when heated but get hardened when cooled. Unlike thermosetting polymers, these polymers can be remolded into different shapes and sizes, and hence they are recyclable. The physical properties of thermoplastic polymers change above their glass transition temperatures. The intermolecular forces within the polymer chains are weak, and the polymers soften when heated. Processing techniques such as compression molding, injection molding, and extrusion are used to shape thermoplastic polymers. Some examples and applications of thermoplastics: polycarbonates are used in electronic components and automotive parts; polyethylene is one of the widely used polymers in this category that has applications in automobile parts, outdoor furniture, water drainage pipes, packing film, and also squeeze bottles; polyvinyl chloride is used for roofing sheets, drain pipes, inflatable production and electrical insulation; and polypropylene is used in reusable plastic food containers, piping systems, insulation for electrical cables; polystyrene is used to make disposable cutlery, insulation, and packing materials (in the form of foam), and its copolymers are used for making toys.

11.2.3 Mechanical Properties

Tensile strengths of polymers are generally very low as compared to metals, but their ductility can be very large. The mechanical properties of polymers are highly dependent on their chemical structures, crystallinity, and ambient temperature. Cross-linking of polymer chains increases the tensile strength. Also, the length of polymer chains contributes to the tensile strength. Crystalline polymers have higher tensile strength than amorphous polymers since crystalline polymers are closely packed, facilitating secondary bonding. Polymers behave differently with change in temperature, for example, brittle at very low temperatures and highly ductile at relatively high temperatures.

Similar to strain-hardening in metals, predeformation of polymers by drawing (mechanical deformation) increases the tensile strength due to proper alignment of polymer chains. In fact, deformation of polymers involves necking, which makes the material stronger for similar reasons as in metals. The strain rate at which the polymers are deformed also has an effect on their ductility. Polymers behave more ductile when the strain rate is decreased. *Crazing* is a phenomenon which occurs in glassy thermoplasts before brittle fracture. This increases the fracture toughness of polymers, since high localized yielding occurs during the phenomenon that facilitates formation of interconnected microvoids.

Polymer fracture process involves three stages: (1) crack initiation, (2) crack propagation, and (3) failure. The crack initiation stage can be small scratches or defects on the surface. Mechanistically, fracture in polymers involves breaking of covalent bonds in the polymer chains. Depending of the polymer type and environmental conditions (temperature and chemical presence), the fracture can be ductile or brittle.

11.2.4 Degradation

Polymers may undergo degradation in long-term exposure to heat, light, or chemicals. Degradation significantly affects the properties of polymers. It leads to the reduction in chain length or molecular weight and consequently causes failure or cracking of polymers. The pH of the environment plays an important role in the degradation of a polymer. For example, acidic and alkaline media are more aggressive than neutral solutions. Chloride ions in water may cause chloride-induced cracking in plastic pipes. Degradation of polymers is also dependent on their chemical structures. The main mechanism of degradation in polymers is through scission (breaking of bonds) of polymer chains. They are divided into two types: (1) random scission and (2) specific scission. In random scission, the breaking of bonds occur randomly (for example, polyethylene), whereas in specific scission the breaking of bonds occurs at the chain end and forms monomers (for example, poly (α,-methylstyrene)).

Degradation in polymers generally occurs via hydrolysis, UV light, or a thermal source. Step-growth polymers degrade by hydrolysis in acid or basic aqueous environments and produce low molecular weight molecules. Polyamide and polyethylene undergo degradation in acidic solutions, whereas polycarbonate is reactive to alkaline solutions. The degree of crystallinity of the polymer affects the chemical degradation behavior, meaning amorphous polymers are less resistant to chemical attack as compare to crystalline polymers. Polymers with epoxy or aromatic groups are prone to UV degradation. Polyethylene, poly(methyl methacrylate) and polypropylene undergo UV degradation and oxidation. At high temperatures, polymers may undergo degradation in the presence of oxygen. Hence, it is critical to avoid oxygen in the environment during processing. Chain-growth polymers are not stable at high temperatures. They degrade and produce monomers, gases, oils, and water. Polyvinyl chloride may produce hydrogen chloride gas and turn brittle. Inorganic polymers (silane-based) are more resistant to thermal degradation as compared to organic polymers (carbon-based).

Microorganisms in soil or water may cause degradation in polymers, which is known as biodegradation. The polymers which are susceptible to biodegradation are called biodegradable plastics. Basically, oxygen and moisture are required for biodegradation of plastics. In fact, biodegradable polymers are sometimes desirable when they are dumped as waste in landfills, preventing environmental pollution.

11.3 Ceramics

Ceramics are inorganic, nonmetallic materials that are composed of metals and nonmetals. Alumina (Al_2O_3), silica (SiO_2), and silicon carbide (SiC) are examples of ceramics. Alumina is one of the most commonly used ceramics in industries. Traditional ceramics include clay-based materials, such as bricks, porcelain and tile, and glasses. Ceramics generally have high melting points, ranging between 1,000 °C to 1,800 °C. The density of ceramics is between that of metals and polymers. Ceramics are generally electrical and thermal insulators that are brittle in nature. They attain different colors by adding pigments. The ions in the ceramics also contribute to the color. Ceramics are predominantly anisotropic, which means the mechanical properties are directionally dependent; in other words, they have different mechanical properties in different directions. They are widely used in domestic, industrial, and building products. Advanced ceramics have applications in semiconductor (electronics) and aerospace industries. There are various techniques to produce ceramic products, depending on the structure (crystalline or noncrystalline) of the ceramics. For crystalline ceramic materials, the two common routes used for making products are (1) in situ reaction and (2) sintering powder. The techniques involve injection molding, dry pressing, and slip casting. In the case of noncrystalline ceramic materials, the products are made from the melts.

11.3.1 Chemistry

Bonding in ceramics is generally mixed, a combination of ionic and covalent bonding. The degree of ionic bonding depends on the electronegativity difference between the cations and anions, for example, calcium fluoride (CaF_2), silica (SiO_2) and silicon carbide (SiC) exhibit 89 percent, 51 percent, and 12 percent ionic bonding, respectively. Highly ionic crystals contain alkalis or alkaline earths as cations and oxygen or halogens as anions. The type of bonding in ceramics determines the physical and mechanical properties. Ceramics are generally hard and brittle and possess poor electrical conductivity (insulators). They are predominantly amorphous (for example, glasses); however, there are crystalline and semicrystalline ceramics as well. Noncrystalline ceramics have glass transition temperatures, whereas crystalline ceramics have melting points. For a ceramic containing both crystalline and noncrystalline phases, when the temperature is above glass transition temperature, the noncrystalline phase will be ductile and the crystalline phase will be brittle, whereas when the temperature is above melting point then both the phases will melt. Crystalline ceramics are denser than noncrystalline ceramics. Some of the popular crystalline ceramics are listed in Table 11.2.

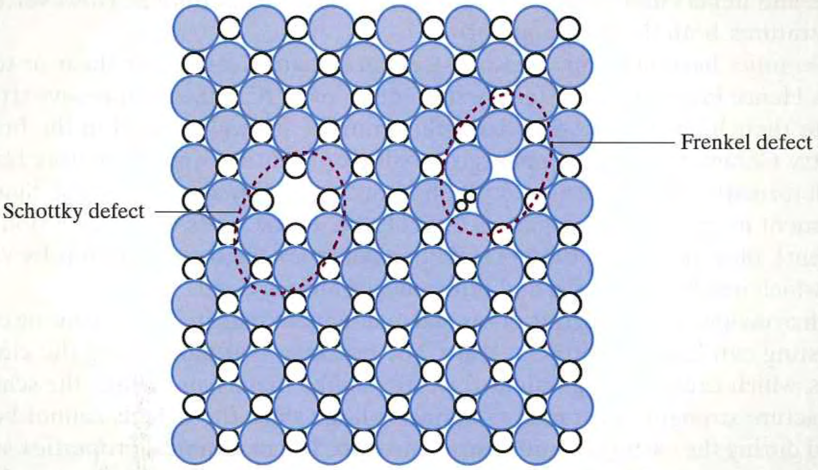
Imperfections in ceramics occur due to the imbalance in charges. Large coulombic forces due to the unbalance charges will create defects in ceramics to maintain charge neutrality. There are two types of charge neutral defects: (1) *Schottky defect* and (2) *Frenkel defect* (Figure 11.4). In a Schottky defect, vacancies are created in the material due to a missing pair of anion and cation. A Frenkel defect is a vacancy due to a missing pair of anion/cation and the presence of an interstitial anion/cation. Impurity atoms can be deliberately added by the substitution of an anion/cation pair or into an interstitial space in the lattice if that maintains the charge of the material. Ceramics are chemically stable and do not easily under corrosion/degradation. They are stable in weak acids and bases but may undergo ion exchange reaction in strong acids and bases. For example, hydrofluoric acid is an aggressive corrosive environment for ceramics.

Ceramics can be broadly classified into two categories: (1) oxides such as silica, alumina, zirconia and ceria; and (2) non-oxides such as carbides, nitrides, and borides. Silicate ceramics are mainly composed of silicon and oxygen. The basic building block of silicate ceramics is SiO_4^{4-} (with Si^{4+} as the cation and O^{2-} as the anion), and the bonding between silicon and oxygen is largely covalent (Figure 11.5). Silica (SiO_2) can be crystalline (quartz) or amorphous (glass). In silicate glass, the basic structural unit is the SiO_4 tetrahedron, but other ions such as sodium may also be present. The

Table 11.2 Crystalline ceramics and their crystal structures

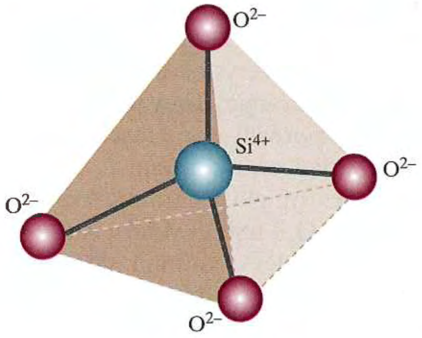
Ceramic	Crystal structure
Cesium chloride (CsCl)	Simple cubic
Perovskite ($CaTiO_3$)	Simple cubic
Fluorite (CaF_2)	Face-centered cubic
Silicates (complex) (SiO_2)	Face-centered cubic
Spinel ($MgAl_2O_4$)	Face-centered cubic
Rock salt (NaCl)	Face-centered cubic
Corundum (Al_2O_3)	Hexagonal closed pack

Two types of imperfection in ceramics **Figure 11.4**



(Based on Callister W. D., and Rethwisch D.G. 2010)

Crystals structure of silicate **Figure 11.5**



addition of Na_2O (known as soda) reduces the melting point since it breaks the tetrahedral network. However, making glass bottles becomes easier due to soda addition. Window glass mainly contains CaO and Na_2O that are incorporated into the SiO_4 network. High-performance ceramics such as silicon carbide, boron carbide, and tungsten carbide have high hardness and wear resistance. They have applications in ball bearings, ballistic armored vests, and heavy-duty minerals-processing equipment, for example, crushers that are used to reduce the size of large mineral particles for metal extraction.

11.3.2 Mechanical Properties

Ceramics are hard, brittle materials with poor ductility. They can undergo cracking or failure under sudden loading conditions. Temperature and degree of crystallinity of ceramics affect their properties, such as increased modulus and brittleness when the crystallinity is high and the temperature is low. In fact, crystalline and noncrystalline phases respond differently to change in temperatures. Crystalline phases are brittle, and noncrystalline phases are ductile at high temperatures. However, at low temperatures, both the phases are brittle.

Ceramics have high compressive strength but are weak under shear or tension forces. Hence in ceramics, the tensile strength is lower than the compressive strength. Due to their high compressive strength, ceramics are widely used in the building industry. Ceramics in general undergo very little plastic deformation before fracture. The deformation mechanism in crystalline ceramics is by slip movement. Since slip movement in crystalline ceramics is difficult due to the types of bonding (ionic and covalent), they are brittle. In noncrystalline ceramics, the deformation is by viscous flow, which involves breaking and reforming of atomic bonds.

Microscopic defects such as pores and microcracks that are formed during ceramics casting can lead to brittle fracture. The cracking is usually along the cleavage planes, which causes transgranular fracture. Unlike metals and alloys, the scatter in the fracture strength in ceramic materials is large since the defects cannot be controlled during the casting/manufacturing process. The mechanical properties, such as flexural strength and modulus of elasticity, of some common ceramics are given in Table 11.3.

11.3.3 Glass-Ceramics

Glass-ceramics are formed by controlled crystallization of base glass. Nucleation agents are added to control the crystallization process. These ceramics generally have an amorphous phase and one or more crystalline phases. The crystallinity generally ranges from 30–90 percent. The polycrystalline glass-ceramic materials have some unique properties of glasses and ceramics. They possess high strength and toughness, high-temperature stability, zero porosity, good machinability, high-chemical durability, superconductivity, and low dielectric constant.

There are different types of glass-ceramic systems, for example, $Li_2O-Al_2O_3-nSiO_2$ (LAS), $MgO-Al_2O_3-nSiO_2$(MAS), $ZnO-Al_2O_3-nSiO_2$(ZAS). The LAS system mainly consists of oxides of lithium, silicon, and aluminum. Chemicals such as Na_2O, K_2O, and CaO are also added as glass-phase forming agents. This system has many commercial applications due to their thermomechanical properties, meaning that they have high strength and can withstand high temperatures. The LAS system has very low heat conduction coefficient, ideal for cooktops. They are also transparent for IR radiation heating.

Table 11.3 The mechanical properties of some common ceramics.

Material	Flexural strength (MPa)	Modulus of elasticity (GPa)
Aluminum oxide (Al_2O_3)	275–700 (2,750-7,000 ton/ft^2)	393 (393×10^4 ton/ft^2)
Fused silica (SiO_2)	110 (1,100 ton/ft^2)	73 (73×10^4 ton/ft^2)
Silicon carbide (SiC)	100–820 (1,000-8,200 ton/ft^2)	345 (345×10^4 ton/ft^2)
Silicon nitride (Si_3N_4)	250–1000 (2,500-10,000 ton/ft^2)	304 (304×10^4 ton/ft^2)
Soda-lime glass	69 (690 ton/ft^2)	69 (69×10^4 ton/ft^2)
Zirconia (ZrO_2)	800–1500 (8,000–15,000 ton/ft^2)	205 (205×10^4 ton/ft^2)

11.4 Composites

Composite materials contain two or more of the basic materials: metals, ceramics, and polymers. The two main constituents in these materials are matrix and reinforcement. The purpose of the matrix is to support the reinforcement material, which is generally in the form of particles or fibers. The reinforcement material will provide the desired mechanical properties to the composite. Composites are made-to-order materials that possess different physical or chemical properties than the individual basic materials. The synergetic effect overcomes the weakness of one material by the other. Composite materials are generally stronger and lighter as compared to traditional materials.

Composite materials can be broadly classified into three categories: (1) polymer matrix composites, (2) metal matrix composites, and (3) ceramic matrix composites. The other special types of composites are sandwich-structured composites and high-strain composites. In sandwich-structure composites, a lightweight, thick material with low strength is used as a core material, and it is covered with thin materials with high bending stiffness. Polymers such as polyethylene, polyvinylchloride, or polyurethane are also used in the form of foam (open and closed cell structure) for the core material. High-strain composites are produced for structures requiring high flexibility.

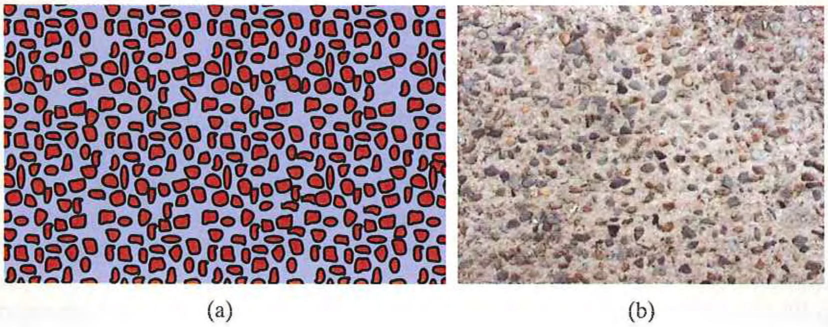
Particle-reinforced composites contain particles distributed evenly in a material matrix (Figure 11.6). The process of making these composite materials is easy and cheap, for example, gravel in cement. These composites can have good strength, stiffness, and toughness, but more importantly they have high wear resistance. However, compared to fiber-reinforced composites, particle-reinforced composites are inferior in terms of mechanical properties.

11.4.1 Fiber-Reinforced Plastics

Fiber-reinforced plastics contain two components: (1) matrix material and (b) fibers. The fibers are reinforced in the matrix material. The mechanical properties, such as strength, stiffness, and elasticity, can be tailored by altering the alignment of fibers to suit specific design requirements.

Figure 11.6

(a) Schematic representation of particle-reinforced composite, and (b) a photograph of gravel in cement



(a) (b)

(Courtesy of M. Bobby Kannan, James Cook University, Australia)

Matrix

Polymers are commonly used as matrix materials, also called binders, in composites. Polyester, epoxy, polypropylene, and polyimides are some the commonly used binders. The mechanical strength of the composites depends on the ratio of matrix material and fiber material (reinforcement). Composite materials typically contain approximately 60 percent matrix. The purpose of the matrix is to hold and cover the fibers, transfer load to and between fibers, and prevent contact of the fibers with an aggressive environment. The matrix should also have physical and chemical stability during the manufacturing process of the composite. Under loading condition, the soft matrix may crack, and as a result, debonding of fiber and matrix can occur. However, the orientation of the fibers is not disturbed and hence the fibers are expected to take over the load and avoid cracking or failure.

Thermosets and thermoplastic polymers are used as matrix materials in composites. Thermosets such as epoxies, polyurethanes, and polyimides are widely used. Epoxy polymers are transparent, and their composited materials are used in aerospace, sporting equipment, chemical industries (pipes), and printed circuit boards. They are expensive but have better resistance to moisture. Polyimides matrix composites can be used in extreme climatic conditions, such as hot temperature or wet conditions. Thermoplastic polymers, such as polyphenylene sulphide and polyetherimide, withstand different conditions (hot/wet) and also have high impact resistance. Polyester-based composites are used in surfboards, but the polymer is susceptible to UV degradation. Vinylester is relatively transparent and more resistant to degradation than polyester. Both polyester and vinyl ester are widely used as matrix materials reinforced with glass fibers. Their applications include automobiles, bathtubs, pipes, and reactors.

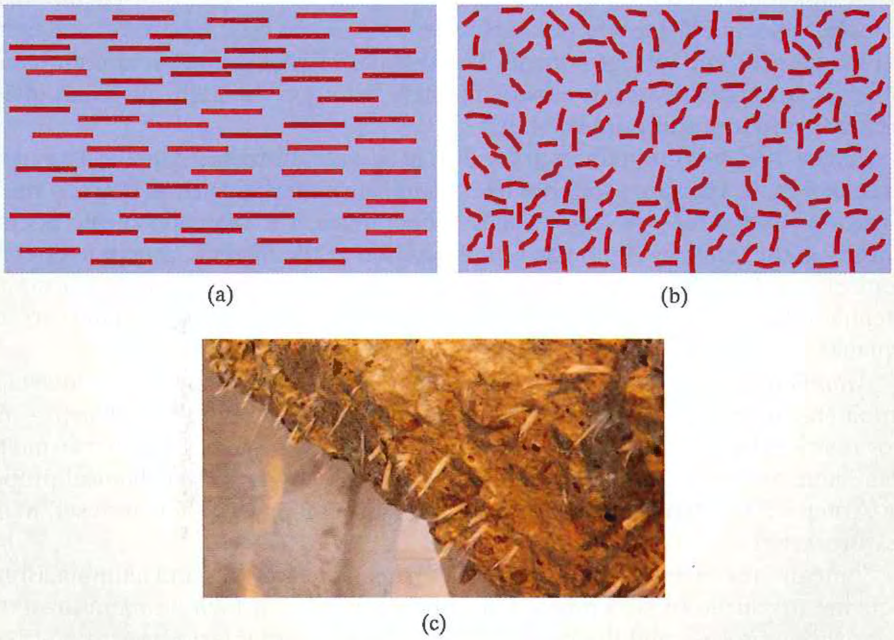
Shape memory polymer composites contain shape memory polymers such as epoxy-based, acrylate-based, and cyanate-ester-based polymers as matrix materials. These composites respond to heat as they possess high strength and stiffness at low temperatures and become soft above their glass transition temperatures. Hence, these composites can be shaped or reshaped by heating, importantly without loss in their physical or mechanical properties. These composites have applications in deployable structures. Epoxy-based matrix composites have application in outdoor equipment and automobile bodies. Cyanate-based matrix composites are used in space applications. Acrylate-based matrix composites have application in sensors operating at cold temperatures.

Fiber

Reinforcement fibers improve tensile strength and stiffness in composites. They provide rigidity and resistance to crack propagation. There are certain materials requirements for forming a good reinforcement of fibers in a polymer matrix. The fibers should bond chemically with the matrix for better adhesion within the curing time. The reinforcement of the fibers should not be bundled; they should be placed separately such that the failure would not be catastrophic. To protect the fibers from environment degradation or any other damage, such as cuts, the fibers should be completely covered by the matrix. This would also help in transferring the applied load to the fibers.

In fiber-reinforced composites, the fibers can be aligned in a continuous or discontinuous fashion in a matrix (Figure 11.7). The fibers in the form of filaments or bundles solely carry the load and also redistribute the load if individual fibers break,

Figure 11.7 Schematic representation of: (a) continuous fiber-reinforced composite, (b) discontinuous fiber-reinforced composite, and (c) a photograph of plastic fiber-reinforced concrete


(a) (b) (c)

(Courtesy of R. Tuladhar, James Cook University, Australia)

hence, avoid sudden failure. The fibers are long in the continuous fiber composites, and the maximum strength is along the fibers' alignment direction, hence they are anisotropic. As the diameter of the fiber increases, the strength decreases. Thin fibers are advantageous since they can attach well to the matrix and provide high strength to the composite. Continuous reinforced composites generally have layered or laminated structures. In discontinuous fiber composites, the fibers are short and randomly aligned. They are isotropic and have high ductility and formability, but their strength levels are less. Composites with short and long fibers in the form of flakes and chips are generally produced by compression molding and sheet molding.

Orientation of fibers (unidirectional, two-dimensional, or three-dimensional) affects the mechanical properties such as strength, flexibility, and elasticity of composite products. When the applied load is in the direction of the fiber orientation, the composite material will exhibit high resistance to deformation or fracture. However, when the load is perpendicular, the materials may fail. To avoid this problem, fibers can be orientated in a multidimensional fashion.

Fiber Types

There are various types of fiber materials, which can be used as reinforcement materials in composites. The three main types are: (a) glass, (b) carbon, and (c) aramid.

Glass fibers: Fiberglass-reinforced plastics are commonly known fiberglass. The glass fibers contain compounds, such as SiO_2, Al_2O_3, B_2O_3, MgO or CaO. They are heated at high temperatures and extruded into fibers, typically in the range of 10–15 μm diameter. Long fibers (5–25 mm) and short fibers (~0.3 mm) are used during the molding process. Short fibers are preferred in injection molding for reinforcement in thermoplastic. The most popular glass fiber is calcium aluminoborosilicate glass. Glass-reinforced plastics are used in water and wastewater transportation, for example, pipes and fittings for potable water, desalinated water, seawater and industrial wastewater, and sewage systems. They also have applications in aircraft panels, rocket-motor cases, pressure vessels, and also fishing rods. High silica and quartz fibers are used in specific applications.

Carbon fibers: Carbon fibers are widely used as reinforcement materials in composite materials. They are produced by carbonizing materials such as polyacronitrile or rayon by oxidation/pyrolysis at high temperatures. The diameter of the fibers is typically in the range of 10–15 μm. Graphitization of the fibers is known to improve the mechanical properties. Carbon fiber composites are used in solar panel substrates, antenna reflectors, heat shields (launch vehicles), and disk brakes of racing cars and airplanes.

Aramid fibers: Aramid fiber is an aromatic polyamide, a polymer, produced by combining an amine group and a carboxylic acid halide group. The polymer is also known as Kevlar. It is spun from a solution into a fiber, since the polymer melt is rigid. There are various grades of aramid fibers with a range of mechanical properties. Aramid fibers have high specific strength and toughness. They are used in gas pressure vessels.

Some of the other fiber materials are boron, silicon carbide, and alumina. Boron fibers are produced by decomposition of boron halides at high temperatures. It is an expensive process, and their mechanical properties are inferior to carbon fibers. However, silicon fibers are relatively cheaper and have better properties than boron fibers. Composites with silicon carbide matrix are used in sports and luxury cars.

11.4.2 Metal Matrix Composites

In metal matrix composites, reinforcing materials such as silicon carbide and alumina in the form of particles or fibers are dispersed in a metal matrix like aluminum, magnesium, and titanium. These composites have many attractive properties such as high strength and stiffness, high temperature resistance, creep resistance, and abrasion resistance. The fibers can be reinforced in the metal matrix by continuous or discontinuous fashion; however, the latter is relatively easy to produce. In continuous reinforcement metal matrix composites, wires, or fibers, such as carbon fibers are used, and due to their orientation, they are anisotropic. Discontinuous metal matrix composites generally contain short fibers, whiskers, or particles. Since they are mostly isotropic, they have many advantages. For example, they can be mechanically treated (forged, extruded, or rolled). If the reinforcement materials are not stable, they can be coated with a stable material. High density and production costs are the major drawbacks of metal matrix composites.

There are various methods for producing metal matrix composites, for example, the powder metallurgy route in which powder metal and reinforcements are mixed, compacted, and extruded; foil diffusion bonding in which long fibers are sandwiched with metal foils; electrochemical methods where deposition is made from a solution containing reinforcing particles and metal ions; physical vapor deposition in which vaporized metal is deposited on fibers; and spray deposition in which fibers are sprayed with molten metal.

11.4.3 Ceramic Matrix Composites

It is well known that ceramics are brittle; hence, ceramic matrix composites are mainly produced for better fracture toughness. The fracture toughness can be increased by reinforcing fibers or particles, which make the ceramic matrix composites. These materials have high temperature creep resistance, oxidation resistance, and thermal shock resistance. High sintering temperatures used for traditional ceramics cannot be used for ceramic matrix composites, since the fibers may oxidize at high temperatures. However, the use of the glassy form of ceramics, which softens at low temperatures, could avoid this problem.

11.4.4 Failure

Composites can fail under the tension or compression mode of stress. Under tension, the bonds between the fibers and matrix can break and cause the material to fail. Compression buckling as a whole or at the fiber level can cause failure. Failure in fiber-reinforced composites can occur by many causes, but primarily by delamination, debonding of fiber/matrix, and fiber fracture. Delamination is the separation at the interface between two layers of the composite material, which can occur due to repeated cyclic stress. In this case, the fibers can be separated from the matrix. The composite material will undergo shear at the interface between the fibers and matrix when the tensile forces stretches the matrix more than the fibers, which could lead to fracture of the material. Brittle composites fail quickly after the initial onset of failure, but other composites have high toughness and the crack grows slowly even after the onset of failure. Degradation of the composites can also lead to failures. Moisture absorption tendency, limited ductility, and difficulty in detection of defects/cracks or repair are some of the disadvantages of fiber-reinforced composites and thereby limit the widespread applications of composite materials. Nondestructive techniques such as X-ray radiography, ultrasound, and thermography can be used for testing composite materials.

11.4.5 Applications

Composite materials have widespread applications in the aerospace and aircraft industries, race-car bodies, and also in high-end sports equipment (tennis rackets and skateboards) due to their attractive properties such as high strength and low weight. They also have applications in the building, bridge, and marine construction industries.

Fiber-reinforced plastics are suitable for weight saving. They have found application in aerospace components such as fuselages and wings, bicycle frames, race-car bodies, storage tanks, and swimming pool panels. They also have potential applications in buildings and bridges, for example, strengthening (flexural and shear) beams, columns, and slabs. Waste materials are used as fiber for reinforcement of composite materials in applications such as building materials, outdoor deck floors, fences, and indoor furniture. Disposal and recycling is a major concern with fiber-reinforced plastics, since separation of fiber from matrix is not easy. The matrix degradability should also be considered while producing composites, since all polymers are not degradable. Biodegradable polymers can be a better choice to address environmental concerns.

Metal matrix composites have applications in aircraft components, tank armor (steel-boron nitride), automobile parts, such as disc brakes and drive shafts (aluminum-boron nitride), bicycle frames (aluminum-boron carbide), and power electronics (aluminum-graphite). Depending on the application, the matrix can be a lighter material such as magnesium and aluminum or high-temperature-resistant material such as cobalt.

11.5 Summary

1. Polymers are long-chain molecules containing repeated units of monomers. They are mainly covalently bonded.
2. The degree of polymerization is the length of the polymer chain. As the chain length increases, the physical and mechanical properties, for example, viscosity, glass transition temperature, strength, and toughness of the polymer changes.
3. Polymers can be crystalline and/or amorphous. The crystalline melting point is a transition temperature where a fully crystalline or semicrystalline material is transformed to an amorphous material. The glass transition temperature is when an amorphous polymer undergoes transition from a viscous liquid to an amorphous solid upon cooling.
4. Polymers are classified into two types: (1) thermosetting polymers and (2) thermoplastic polymers. Thermosetting polymers, once cured, become hard and cannot be remolded. However, thermoplastic polymers can be remolded into different shapes and sizes.
5. Polymers undergo degradation via hydrolysis, UV light, or heat. Microorganisms in the soil can cause polymer degradation in the presence of oxygen and moisture.
6. Ceramics are hard, brittle materials made up of metals and nonmetals. They have high melting points and are bonded by ionic and covalent bonds.
7. Ceramics are present in amorphous, crystalline, and semicrystalline form. Crystalline materials have melting points, and noncrystalline materials have glass transition temperatures.
8. Ceramics are classified into two categories: (1) oxides, such as silica and alumina, and (2) nonoxides such as carbides and nitrides.
9. Ceramics have high compressive strength but poor tensile strength. They undergo very little plastic deformation before fracture.
10. Composites contain two or more basic materials (metals, ceramics, and polymers), in the form of matrix and reinforcement materials. Fiber-reinforced plastics, metal matrix composites, and ceramic matrix composites are the three main types of composite materials.
11. Reinforcement materials are generally in the form particles or fibers. Fiber-reinforced composites are more effective in improving the mechanical properties as compared to particle-reinforced composites. Orientation of the fibers affects the mechanical properties, such as strength and flexibility of composites.

Exercises

1. What is a polymer?
2. List the techniques that can be used to characterize polymers.
3. What types of bonding occur in polymers?

4. Define degree of polymerization.
5. Explain the difference between crystalline melting point and glass transition temperature in polymers.
6. Why are plasticizers added to polymers?
7. Explain the differences between thermosetting and thermoplastic polymers.
8. Comment on the degradation behavior of polymers.
9. What is the significance of crystallinity in polymers?
10. Explain the deformation mechanism in polymers.
11. What is a ceramic?
12. Explain the types of imperfections in ceramics.
13. How are ceramic components fabricated?
14. What are glass-ceramics?
15. Comment on the tensile and compressive strength of ceramics.
16. What is a composite material?
17. Explain the purpose of matrix and reinforcement materials in composites.
18. What are the advantages of shape memory polymer composites?
19. What is the significance of fiber orientation in composites?
20. List the types of fibers used in composite materials.
21. What are the advantages and disadvantages of metal matrix composites?
22. What metals are commonly used as matrix in metal matrix composites?
23. Describe the methods used to produce metal matrix composites.
24. What are the advantages and disadvantages of ceramic matrix composites?
25. Explain the failure mechanism in composite materials.

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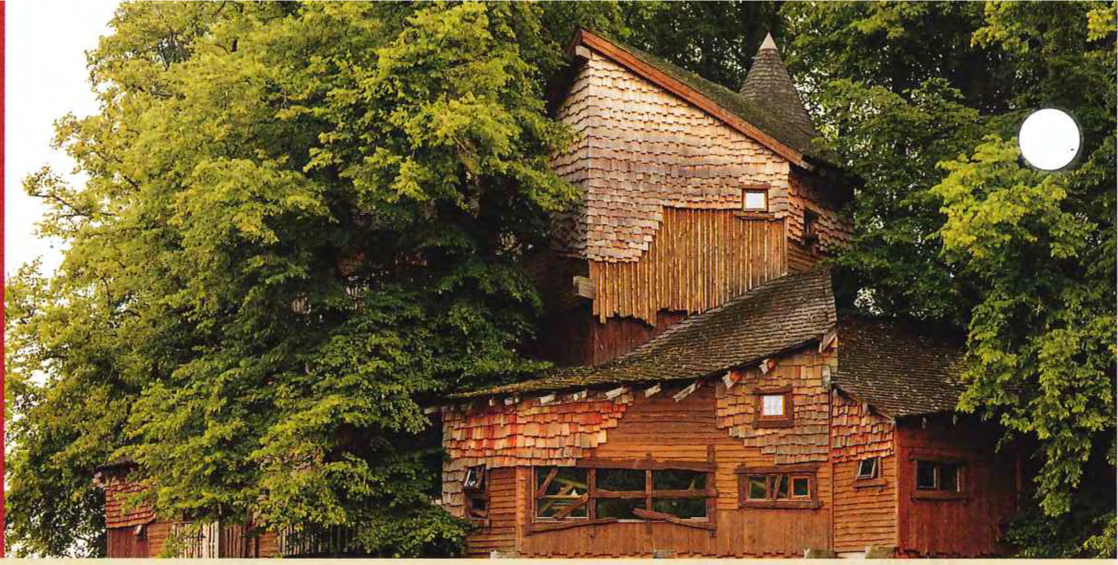
12
CHAPTER



The Alnwick Garden treehouse in Northumberland, England is one of the largest treehouses in the world, built from three different types of sustainably sourced wood. ATGImages/Shutterstock.com

Wood

12.1 Introduction

Wood is one of the oldest building materials used by humans. Due to its wide availability and strength, it has been an important building material since humankind began building houses and bridges. The term *wood* is generally used to refer to hard and fibrous material which forms the trunk, branches, and roots of a tree. In Commonwealth countries, wood that has been harvested and sawn to suitable dimension for use in carpentry and building construction is referred as "timber." In the North America, "timber" usually means felled trees, and "lumber" is used to refer to sawn wood products ready to be used as a building material.

Lumber is used extensively as the structural members for building residential houses, commercial buildings, bridges, and other structures. Due to its strength, versatility, and construction efficiency, lightweight lumber frames and lumber roof trusses are widely used for domestic houses in many parts of the world. Laminated veneer lumber (LVL), glued laminated wood (glulam), and other engineered lumber products are used in large-scale structures, such as industrial and commercial buildings, bridges, and so on. Since long before the advent of steel and concrete, lumber has been used as a chief building material around the world due to its wide availability and aesthetics. The magnificent Todai-ji temple (originally built in 751 CE and reconstructed in 1709 CE) in Nara Japan (Figure 12.1) is considered one of the largest wooden buildings in the world. The Temple of Heaven (Figure 12.2a) in Beijing, China, constructed in 1420, is another example of the architectural beauty of lumber structures. Lumber has also been extensively used in building construction around the world (Figure 12.2b). Lumber is also extensively used as decorative nonstructural members, such as flooring, windows and doors, staircases, railings, and partition walls.

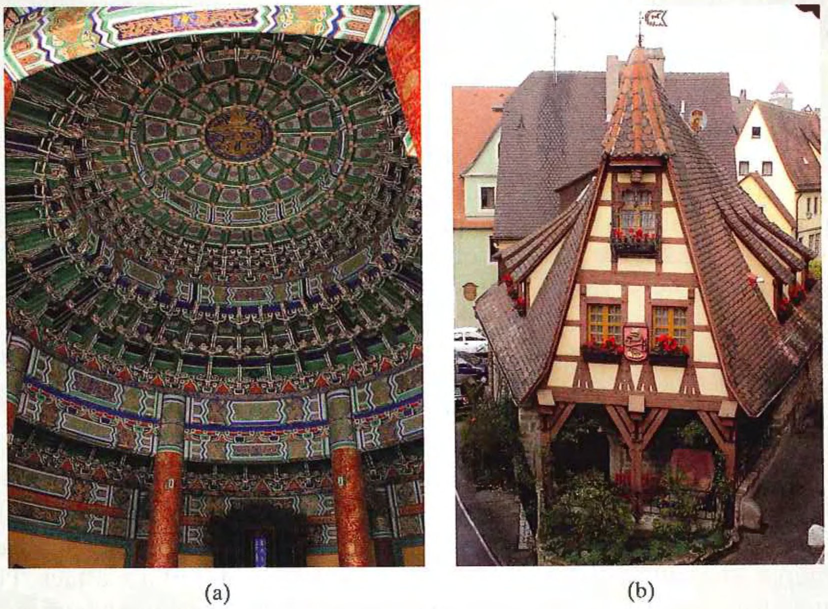
Todai-ji temple in Nara, Japan

Figure 12.1

(Courtesy of R. Tuladhar, James Cook University, Australia)

(a) Inside the Temple of Heaven, Beijing China (b) Timber frame house in Rothenburg ob der Tauber, Germany

Figure 12.2



(a)

(b)

((a) Courtesy of R. Tuladhar, James Cook University, Australia)
((b) Courtesy of Ms. Bhava Dhungana)

12.2 Advantages of Lumber

One of the chief advantages of wood as a construction material is its production efficiency. The production and processing of wood products are highly energy efficient and uses much less energy compared to other building materials like steel and concrete.

When trees grow, they absorb carbon out of the atmosphere. The absorbed carbon is trapped in the wood when they are harvested and is locked within the wood products for their life unless they decay or are burned. Hence, using the wood from trees harvested from well-managed forests helps remove carbon dioxide from the atmosphere. Wood is also the only 100 percent renewable construction material. Trees responsibly harvested from managed forests are replaced by new trees, which further absorb carbon dioxide from the atmosphere and generate oxygen. Wood is a natural building material that is not toxic, and it creates a healthy living environment. It is also biodegradable.

Wood products are also durable if proper measures are taken against degradation. Wood also has a relatively high strength-to-weight ratio, which makes it easier to produce, transport, and erect (Thelandersson and Larsen, 2003). Wood also has very good insulating property, which improves thermal efficiency in houses. Moreover, additional insulation material can be placed in spaces between wood frames without increasing, the thickness of walls, ceilings, or floors. Wood is used extensively for architectural designs for its excellent aesthetic beauty and versatility. Wood naturally has good acoustic properties. Hence, it is also used as claddings, wall linings, and ceilings in theaters, auditoriums, and public buildings to reduce undesirable echoes and reverberations.

12.3 Limitations of Wood

As wood is a natural material, it is vulnerable to biological decay and termite attack if appropriate prevention measures are not taken. At moisture content above the fiber saturation point and in the presence of oxygen, fungi growth can cause wood decay (rot). This can be prevented by controlling the moisture content in wood (Bootle, 2005). Using pretreated wood products or applying wood preservatives can prevent excessive moisture content in wood. Piping in wood is the loss of wood in the center of the member due to decay (Figure 12.3). Piping usually starts at the end grains exposed to moisture and eventually grows to the center of the member. Piping reduces the cross-sectional area of the core of the wood members, significantly affecting its strength and stiffness. Termite attack is another major cause of degradation of wood structures. Appropriate design and construction practice, careful site preparation, preservative treatment of wood, and regular maintenance and inspection are required to ensure protection of wood structures from termite attack. Physical termite barriers, such as concrete slab barriers, stainless steel mesh barriers, and crushed granite barriers are used to physically block termites from attacking wood structures. Chemical barriers can also be used to treat the soil around footings and slabs to inhibit termite attacks.

Wood can also have reduced strength due to the existence of natural cracks and knots. Engineered woods such as laminated veneer wood (LVL), glued laminated wood (glulam), and cross-laminated wood (CLT) products avoid such natural defects and are stronger and more uniform compared to natural wood

Piping and rot in a wood girder

Figure 12.3

(Courtesy of Justin Dewey)

(Section 12.14). Natural wood also shrinks when it loses moisture. The reduction in dimensions of wood due to loss of moisture from the cell walls of the wood is called shrinkage. Shrinkage in wood is greater perpendicular to wood grain and is minimal parallel to the grain. Shrinkage in wood can cause warping, distortion of wood members, and damage in connections. Shrinkage effects are reduced by seasoning the wood (Section 12.9). For unseasoned wood, allowances for shrinkage should be considered in the design process.

Wood is a combustible material. However, if the wood member has sufficient size and has appropriate mechanism to mitigate fire damage, wood structures have good fire resistance. Furthermore, wood is a good insulator and burns in a slow and predictable way: hence wood members can maintain their structural integrity when exposed to fire.

12.4 Structure of Wood

Structural wood used in construction primarily comes from the trunk of a tree. The outermost thin and rough layer of a tree trunk is called bark, and it protects the tree from fire, temperature, and injuries. A microscopic layer of wood cells between the bark and the wood is called cambium. New wood cells are formed continuously on the inside of the cambium, which appears as a ring-like structure called growth rings (Figure 12.4). In summer when the growth is slower, smaller dense wood fibers called latewood or summerwood are formed, resulting in darker rings. On the other hand in

Figure 12.4 Tree trunk section showing growth rings

(Courtesy of R. Tuladhar, James Cook University, Australia)

the spring when the growth is rapid, larger wood fibers with thinner walls are formed which makes lighter growth rings. This wood produced in springtime is called earlywood or springwood. The number of growth rings (annual rings) in a tree approximately represents its age. Also rapid-growing trees have wider rings, and slow-growing trees have thinner, denser rings.

The center of the trunk is called the pith. Heartwood is the inner part of the trunk, which is made up of dead cells. Heartwood is darker, drier, and harder and gives structural support to tree. Sapwood, on the other hand, is the outer region of the trunk containing new, living cells. Sapwood cells act as vertical conduits for conduction and storage of water and nutrients from the roots.

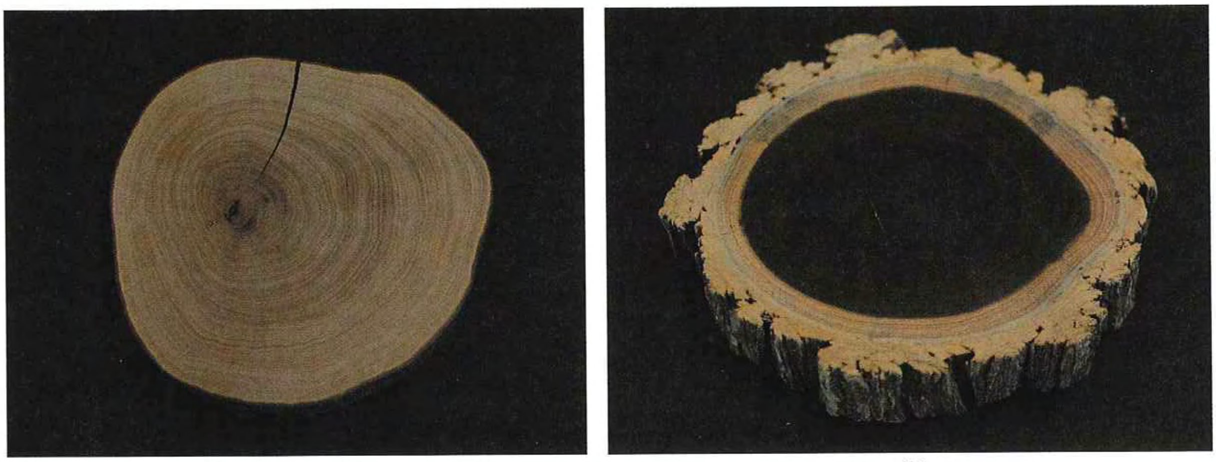
12.5 Types of Wood

Botanically speaking, wood can broadly be classified as softwood and hardwood. However, the terms *softwood* or *hardwood* do not refer to the softness or hardness of the wood. For example balsa is classified as hardwood even though it is one of the lightest and least dense woods.

Softwood (Figure 12.5a) comes from gymnosperm trees. Coniferous trees which have needle-like leaves and produces cone-like uncovered seeds are of this category. Examples of softwood trees are Douglas fir, pine, redwood, spruce, larch, and cedar (Table 12.1). Hardwood (Figure 12.5b), on the other hand, comes from angiosperm trees. Angiosperms are broadleaved trees and produce seeds inside ripening fruits. Examples of hardwood trees are oak, mahogany, maple, teak, aspen, ash, balsa, and hickory (Table 12.1).

(a) Softwood and (b) Hardwood

Figure 12.5



(a)

(b)

(Courtesy of R. Tuladhar, James Cook University, Australia)

Examples of softwoods and hardwoods

Table 12.1

Softwoods	Hardwoods
Douglas fir	White oak
Incense cedar	Ash
Lodgepole pine	Soft maple
Sugar pine	Teak
Jack pine	Aspen
Red wood	Balsa
Yellow cedar	Hickory
Sitka spruce	Hackberry
Balsam fir	Red oak
Tamarack	Black willow
Bald cypress	Tan oak
Atlantic white cedar	Big leaf maple
Larch	Beech

Coniferous trees are usually evergreen, whereas broadleaved trees are deciduous, meaning that they shed leaves in autumn and winter. The main structural difference between hardwood and softwood is that hardwoods have vessels (or pores) which are not present in softwood. Vessels are pipes running along the length of a tree and have large cross-section compared to other wood tissues. They act as conduits of water and nutrients. Softwoods, however, have long, thin cells called tracheids. Tracheids account for 90 percent of the wood volume and provide strength and act as a conduit for water and nutrients.

Most species of hardwoods do tend to have higher densities and are darker in color compared to softwoods. Softwoods generally are easier to cut and work with than hardwoods. Coniferous trees also grow faster than broadleaved trees and are hence generally cheaper than hardwoods. Softwoods account for 80 percent of structural timber used in construction industry around the world. Structural softwood is readily available, lightweight, economical, and easy to use. Structural softwood is widely used in both residential and commercial building frames as studs, joists, bearers, lintels, roof beams, and trusses. Structural softwood is also used to produce engineered products such as glulam and I-beams (Section 12.14). Hardwoods are strong, durable, robust, and beautiful. Hardwoods are mostly used for high-quality furniture, flooring, boats, tools, musical instruments, barrels, and cooking. Hardwoods are used to make large-span floor joists, high-strength structural beams, bridge girders, and wharves.

12.6 Chemical Composition of Wood

Wood is a natural material, and it is made up of cells, which are the smallest organizational units in all living beings. The majority of cells in wood are elongated in shape and have longitudinal axes parallel to the length of the tree. Wood cells consist of a cell wall and a hollow center called the cell cavity. In living cells, these cell cavities help to transport water and nutrients. Wood cells form four types of wood tissues: parenchyma cells, fibers, tracheids, and vessels. Parenchyma tissue transports and stores water and nutrients, whereas fiber tissues give mechanical support to tree. Vessels are found only in hardwoods, and they act as conduits to transport nutrients and water and also provide mechanical support. Softwoods have vertical elements called tracheids, which help to transport water and nutrients.

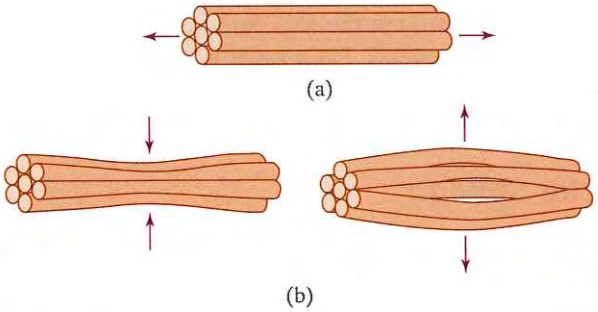
The primary chemical components of wood are cellulose, hemicellulose, lignin, and extractives. These chemicals affect wood fiber properties and the physical properties of the wood. Cellulose is one of the major component (about 40–50% by dry weight) of wood and is composed of linear chains of glucose molecules. Hemicelluloses are branched polymers composed of shorter chains of sugar molecules and help crosslink cellulose microfibrils. Hardwood consists of 15–30 percent of hemicelluloses, and softwood consists of 20 percent. Lignin is the cementing matrix that holds cells together. Softwood has a higher percentage of lignin (23–33%) than hardwood (16–25%). Lignin and cellulose together give mechanical strength to trees. Lignin is hydrophilic compared to cellulose and hemicellulose, which prevents permeation of water through the cell walls and hence helps transport nutrients and water. Lignin also prevents insect and fungi attack on timber. Apart from these three chemical components, wood also contains wide variety of chemical compounds termed extractives. Extractives can be removed from the wood by extraction process using various inert solvents. Different species have varying percentage of extractives (5–30% of dry weight of wood). Some common type of extractives found in wood are carbohydrates, exudates, minerals, and phenolic and nitrogen compounds. Extractives influence the color, density, durability, and flammability of wood.

12.7 Anisotropic Behavior of Wood

Wood is an anisotropic material. It means that the mechanical and physical properties like strength, stiffness, and shrinkage in wood are different in different grain directions. Anisotropy in wood is due to the tubular nature of its cells. Wood cell structure

(a) Straw bundle is strongest parallel to its longitudinal axes. (b)Straw bundle is weakest perpendicular to its longitudinal axes.

Figure 12.6



(Photo: Rabin Tuladhar)

is analogous to bundle of parallel straws bound together by weak glue (Figures 12.6a and 12.6b). Straws here represent the fibers or grain of the wood. The bundle of straws is stronger in tension or compression if the force is applied parallel to the longitudinal axis of the straw bundle (Figure 12.6a). However, straws will tend to crush or pull apart if compression or tension forces are applied perpendicular to the longitudinal axes of the bundle, respectively (Figure 12.6b). Similar to straw, wood cells are also long and tubular in shape with hollow centers. Tubular wood cells are oriented parallel to the longitudinal axis of tree trunk. Wood is strongest when force is applied parallel to directions of tubular cells or grains, whereas wood is weak when force is applied perpendicular to the direction of grain. The tensile strength of wood is about 30 to 50 times smaller in perpendicular to grain than parallel to grain, whereas the elastic modulus is 50 to 80 times smaller in direction perpendicular to grain compared to the direction parallel to grain (Thelandersson and Larsen, 2003). Shrinkage or swelling of wood due to loss or gain of moisture is prominent in the perpendicular direction to the grain as the loss or gain of moisture affects cell walls. The cross-sections of wood members are affected by the change in moisture content. However, the shrinkage or swelling of wood with the change in moisture content is negligible parallel to the grain. The weakness of wood members in the direction perpendicular to the grain should be accounted for in the design and detailing as most of the failures in wood structures are associated with the tension failure perpendicular to grain.

12.8 Conversion and Processing of Wood

Conversion and processing of wood includes harvesting wood from a forest and processing of tree logs and turning them into usable boards and planks. Once the tree is felled, bark, branches, and leaves are removed prior to processing. Clean trunks are then sawn in sawmills to convert round wood into square or rectangular sections. This process is called conversion. If required, wood products are then seasoned to remove moisture from the wood. Different sawing patterns are used to maximize the number of wood products. The most commonly used sawing techniques are (a) flat or plain sawing, (b) quarter sawing, and (c) radial or rift sawing. Each of these sawing techniques produces distinct appearance and characteristics in the end products.

12.8.1 Flat or Plain Sawing

Flat or plain sawing is the most economic and simplest way to cut the log and maximize the wood from the log. Wood is sawn as shown in Figure 12.7a by cutting the log with each slice parallel through the log. Wood produced with plain sawing may twist, bow, warp, or shrink unevenly. Plain-sawn wood is mostly used only for fence construction, formwork, and construction works. Flat-sawn wood also has a tendency to absorb more water than rift-sawn or quarter-sawn wood.

12.8.2 Quarter Sawing

Quarter sawing is done by cutting the log into four quadrants through the log center as shown in Figure 12.7b. Each quadrant is subsequently sawn at right angles to each other, producing planks with the tree's growth rings parallel to the short face. Quarter-sawn timber products are less vulnerable to cupping, distortion, and warping. They are used mostly for floor boards and expensive furniture as it provides a good wearing surface. It is, however, more expensive than plain-sawn timber as it produces less usable timber. Also these timber products are slower in seasoning. Nailing on the face of quarter-sawn timber can also cause splitting.

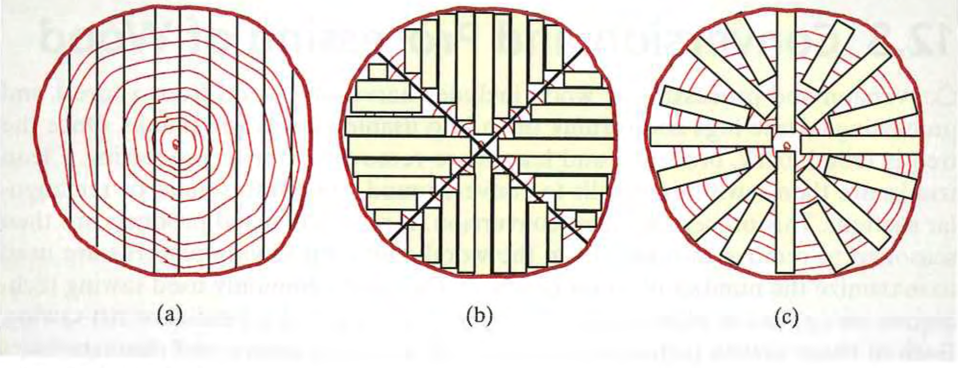
12.8.3 Rift Sawing

Rift-sawn timber (Figure 12.7c) is produced by milling logs radially so that the growth rings are nearly perpendicular to the face. Rift-sawn timbers are stable, have better structural integrity, and are less prone to shrinkage. However, this sawing technique is the most expensive compared to quarter-sawing and plain-sawing techniques because it produces a lot of waste.

12.9 Seasoning of Wood

Freshly felled trees contain a large moisture content (40–50%). Wood that has not been dried and has a high moisture content (>25%) is called "green wood" or unseasoned wood. Green wood is vulnerable to warping, decaying, and shape changes due

Figure 12.7 Sawn timber: (a) Plain-sawn (b) Quarter-sawn (c) Rift-sawn



(Photo: Rabin Tuladhar)

to moisture loss. Seasoning is the process of drying wood to equivalent moisture content (EMC) appropriate for its use. Seasoning helps to prevent shrinkage to enhance dimensional stability of wood members and enhance their mechanical properties like strength and stiffness. It also helps to minimize the splitting and warping of wood members. Seasoned woods are also less prone to decay and fungal attack. It is also easier to apply preservatives, paint, and glue on seasoned wood. Seasoning can be done by air drying, kiln drying, or chemical seasoning.

a. Air drying: Air drying is a process of seasoning wood naturally by exposing it to air. Wood members are dried by stacking them in the open air, using "stickers" or "strips" to ensure vertical and horizontal separation between each members so that ambient air can circulate around each piece. The end of the wood can be painted to retard fast loss of moisture through end grains. A waterproof covering is often placed over the stack to protect wood from direct exposure to the elements. This method requires no additional energy or expensive equipment. However, it requires a large space and takes a considerably long time to dry the wood.
b. Kiln drying: Kiln drying uses a controlled environment and external energy to accelerate the seasoning of wood. Hot air and steam are circulated through stacks of wood inside kilns where heat, humidity, and the amount and duration of air can be controlled for optimum drying. The process is much faster than natural drying, typically taking only 20 to 40 hours to dry wood to 12 percent moisture content. Other advantages of kiln drying over air seasoning are that it eliminates fungi and insects as well as reducing more weight.
c. Chemical seasoning: The chemical seasoning process consists of immersing green wood in hygroscopic (that is, moisture-attracting) chemicals to absorb moisture out of wood. One example of chemical seasoning is immersing wood in a salt (sodium chloride) solution, also known as salt seasoning. Other dehydrating agents commonly used in chemical seasoning process are urea, polyethylene glycol, and sodium nitrate. After wood is removed from hygroscopic solutions, wood is seasoned in a normal process like air seasoning. Chemical seasoning is used to produce high-market-value products which are required to be free of surface checks (small surface cracks).

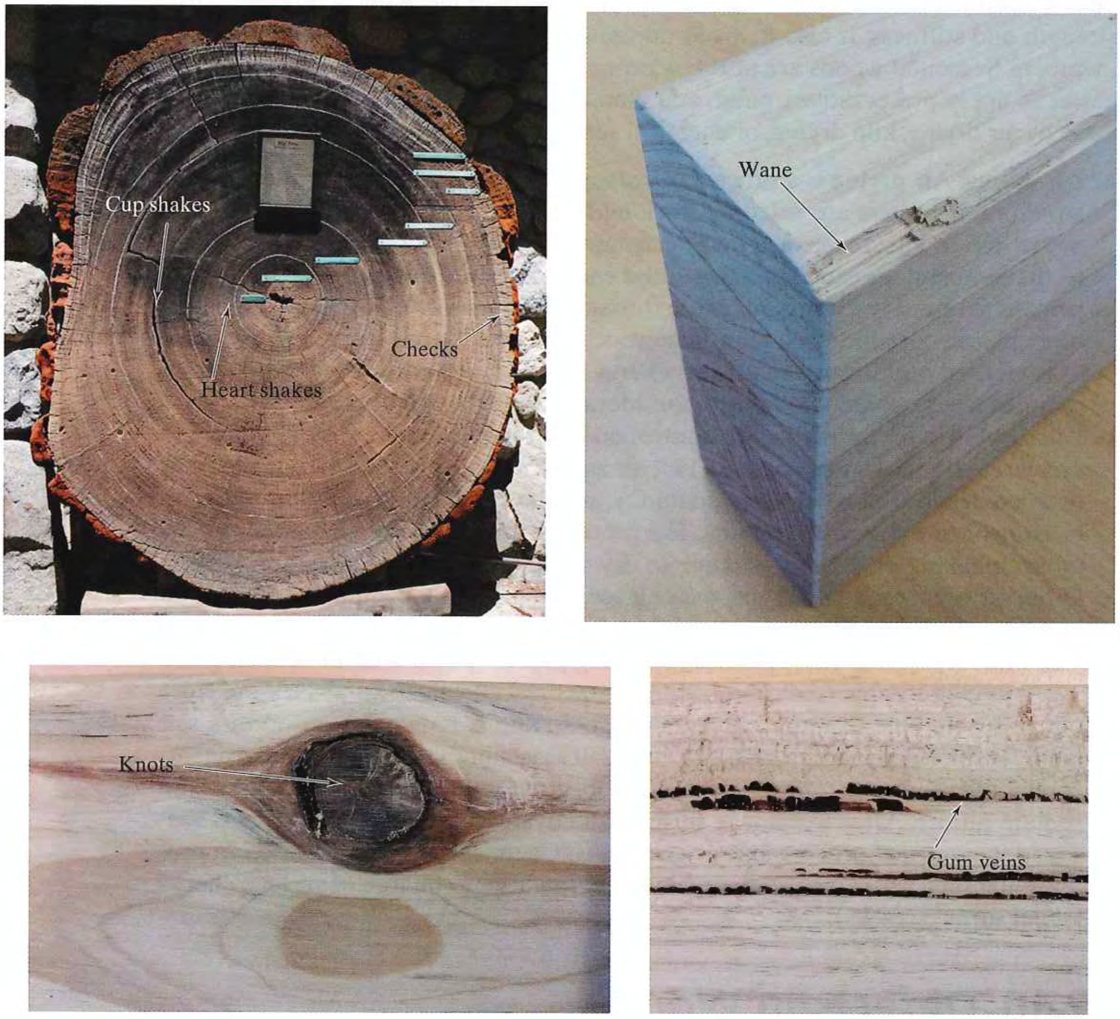
12.10 Defects in Wood

Wood is a natural material, and it can come with various natural irregularities or defects (Figure 12.8). Defects in wood can significantly reduce its strength and durability and hence should be considered in design and construction. Some common natural defects that develop during the growth of tree are knots, checks, cracks, splits, shakes, wane, and gum and resin veins.

Shakes: A shake is a separation of wood fibers along the grains and is often found in older trees. Star shakes extend from the bark toward the center of the trunk. These are mainly formed due to stresses in standing trees. Heart shakes on the other hand extend from the center of the tree trunk to the sapwood and are mainly caused by shrinkage of the interior wood. Cup shakes occur between growth rings, splitting one ring from the other. Cup shakes may develop due to nonuniform shrinkage of wood or due to excessive bending of trees caused by strong winds.

Knots: Knots are the remnants of branch bases that are embedded in the tree trunk. The formation of knots deviates or creates discontinuity in wood fibers. Knots are not only unsightly but also significantly reduce the strength and stiffness of wood.

Figure 12.8 Examples of defects in wood



(Courtesy of R. Tuladhar, James Cook University, Australia)

Checks: Checks are fine cracks appearing on the surface of wood due to uneven drying or shrinkage in radial and tangential directions. Checks do not normally affect the strength of wood.

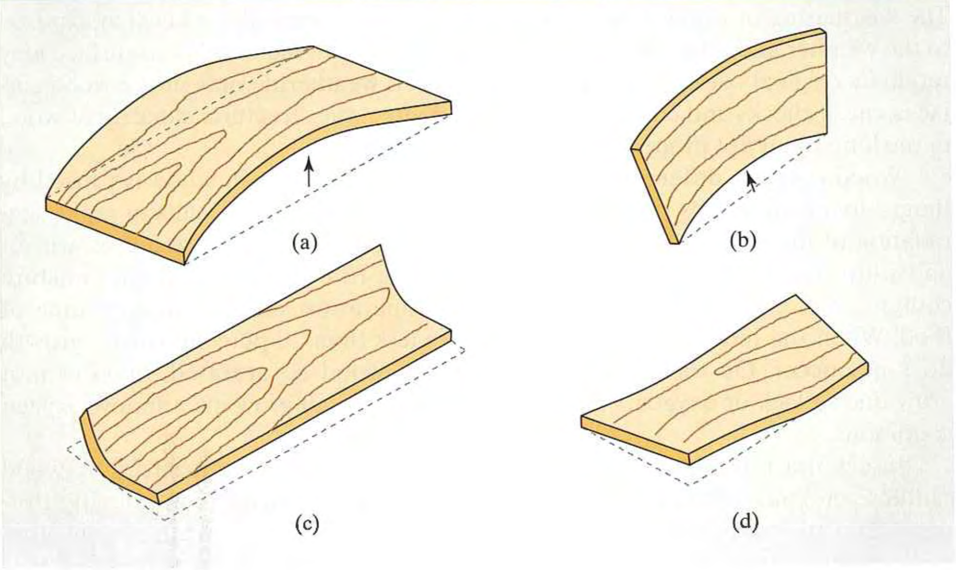
Splits: Splits are checks extending through the full width of wood and are caused by aggressive drying of wood.

Gum and resin veins: A gum and resin vein is the build-up of resin or tree sap. Gum veins are common characteristics in most eucalypt hardwood species like Spotted Gum. Gum is a natural exudation produced by a tree as a defense mechanism against damage caused by insects, fire, or mechanical damage. Gum veins can affect the strength of wood members when gum deposits are loose and do not contain wood fibers.

Wane: Wane is the absence of the outer part of wood on any surface or edge of wood. Wane rarely affects the bending strength of wood but may affect bearing strength.

Warps in lumber pieces: (a) bow (b) spring (c) cup (d) twist

Figure 12.9



(Photo: Rabin Tuladhar)

Some examples of defects in wood that develop during its conversion and processing or during its service life are bow, spring, cup, and twist. Any kind of such distortion in wood member is called warping.

Bow: Bow in wood is characterized by a curvature in the longitudinal direction of the wood plank (Figure 12.9a).

Spring: Spring in wood is a curvature in a wood plank in the plane of its wide face (Figure 12.9b).

Cup: This defect is characterized by a concave curvature of wood across its width in the transverse direction (Figure 12.9c).

Twist: Twist is a spiral distortion of a wood member along its length (Figure 12.9d).

12.11 Degradation of Wood and Preservative Treatment

Wood is a natural material, and when subjected to repeated wetting and drying cycles, it may degrade due to biological attacks and weathering. Wood is biodegradable; hence the discarded wood will degrade with time and is, therefore, good for natural recycling. However, when used as construction material, wood needs to maintain its resistance to degradation and maintain its properties and integrity throughout its service life. The risk of degradation and decomposition of wood is influenced by wood species, presence of moisture and insects, presence of sapwood, and the type of structure where the wood is used.

12.11.1 Degradation of Wood

The weathering of wood is the change in color and texture of wood when exposed to the weather and ultraviolet (UV) light. Wood beneath the weathered surface may retain its original color and properties. However, weathering may erode wood surfaces, cause checks and cracks and warps, and affect the structural capacity of wood in the long run if not properly maintained.

Wood decay is different than weathering. Decay or rot in wood is caused by fungi—living organisms that grow in moist conditions—and it results in rapid degradation of the wood. Most common type of fungi that decay wood are brown rot and white rot. Conditions required for decaying to occur in wood are moisture content over 20 percent, oxygen, suitable temperature, and adequate source of food. When the moisture content of wood is less than 20 percent, fungal growth does not occur. On the other hand, when the wood is saturated, fungi cannot grow due to lack of oxygen. However, soft rot fungi can grow even in low oxygen conditions.

Insect attack is another cause of degradation of wood. Termites feed on wood cellulose, creating extensive galleries within wood and destroying it very rapidly. Borers, such as the pin-hole borer, lyctid borer, and anobium borer, can attack unhealthy and weakened trees. Marine borers can eat wood as food or create holes for their shelter and are particularly destructive to wood structures. For marine structures, it is important to understand the hazard type and apply a suitable chemical preservative treatment and mechanical barriers to prevent damage due to marine borers (Bootle, 2005).

12.11.2 Protective Coatings and Preservative Treatments

Protective coatings such as paint, stains, and water repellents are commonly applied on wood members to protect them from weathering. These protective coatings help inhibit water penetration into the wood and also reduce the weathering due to ultraviolet (UV) radiation from the sun. Protective coatings also help to stabilize wood members against shrinkage and expansion of wood members caused by fluctuation in wood moisture content.

Termite attack on wood can be prevented by proper consideration during design and construction of the structures, application of physical barriers such as stainless steel mesh beneath the concrete foundations supporting wood structures, minimizing direct contact of wood with soil, and chemical barriers such as application of chlorpyrifos and bifenthrin (Bootle, 2005).

At industrial scale, chemical treatments are used to prevent and inhibit degradation of wood and hence enhance the durability and service life of wood products. Preservative treatment of wood consists of impregnating wood cells with chemicals that protect the wood from organisms like fungi and insects, fire damage, and weathering. Pressure and vacuum treatments are often used in preservative treatment to inject chemicals into wood cells. The most commonly used chemicals for preservative treatments are oil-borne preservatives (such as creosote), water-borne preservatives (such as chromated copper arsenate (CCA), ammoniacal cooper quaternary, copper azole, and boron-based compounds), and pentachlorophenol (penta).

Chemicals, such as monoammonium phosphate, diammonium phosphate, borax, and sodium fluoride, are often used to improve fire resistance of wood.

12.12 Physical Properties of Wood

Density and moisture content are closely related physical properties of wood that have significant influence on its mechanical properties, such as strength and durability.

12.12.1 Density and Specific Gravity

The density of wood is its unit weight and is expressed as mass per unit volume of wood. ASTM defines wood density as the ratio of an oven-dry mass of wood (m_o) to the volume of green wood (V_s) (ASTM D2395). Density is often expressed at 12 percent moisture content, which is the typical moisture content for lumber in use. Specific gravity, on the other hand, is the ratio of the density of wood (ρ_s) to the density of water at 4 °C (ρ_w).

$$\textit{Wood density} = \frac{\textit{oven-dry mass of wood } (m_o)}{\textit{volume of green wood } (V_s)}$$

$$\textit{Specific gravity of wood} = \frac{\rho_s}{\rho_w} = \frac{m_d}{V_s} \times \frac{1}{\rho_w}$$

Wood's density or specific gravity depends on wood species, cell size, thickness of cell wall, cell density, the log cut, and moisture content. Freshly cut green wood has higher density than dry wood. The volume and hence the density of wood, varies with its moisture content. The dry density of wood ranges from 300 to 700 kg/m^3 (18-44 lb/ft^3). The strength and stiffness of wood increases with the increase in specific gravity.

EXAMPLE 12.1

The dimensions of a wood sample are 5 × 10 × 15mm and the oven-dry weight of the sample is 0.52 kg. If the moisture content in the sample is 15 percent, calculate the specific gravity of the wood sample.

Solution:

$$\textit{Green volume of wood } (V) = 5 \times 10 \times 15 = 750\ mm^3$$

$$\begin{aligned}\textit{Specific gravity of wood} &= \frac{\rho_s}{\rho_w} = \frac{m_d}{V_s} \times \frac{1}{\rho_w}\\ &= \frac{0.52}{750 \times 10^{-6}} \times \frac{1}{1000}\\ &= 0.69\end{aligned}$$

12.12.2 Moisture Content

Moisture content is another physical prosperity of wood that has a significant effect on its strength and stiffness. Moisture content is expressed as the moisture content of wood in terms of its oven-dry mass, in percentage.

$$\textit{Moisture content } (\%) = \frac{m_s - m_0}{m_0} \times 100\%$$

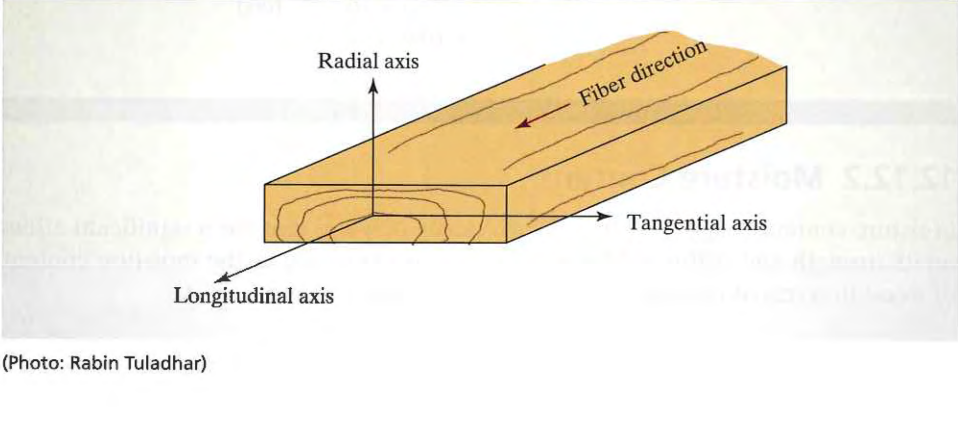
where m_s is the mass of wet wood, m_0 is the mass of wood after oven-drying at 100–105 °C.

Wood in a living tree typically has 200 percent moisture content, which may vary with the species, age, and size of the tree. Water is present in the cell cavities (called free water) or within the cell walls (called bound or absorbed water). When a tree is felled, it starts to lose free water first. Wood ready for manufacturing has a moisture content of about 50 percent. At the fiber saturation point (25–30% moisture content), cell cavities have lost all the free water, but the cell walls are still saturated with the bound water. When the wood is further dried, it starts to lose absorbed water from the cell walls as well. Above the fiber saturation point (FSP), loss of free water does not change the volume of wood and its mechanical properties. However, density is decreased with the loss of free water. When the wood is dried below the fiber saturation point (FSP), absorbed water is driven out from the cell walls, which will reduce the cross-sectional dimension of the wood. Below the FSP, wood will absorb moisture from the atmosphere or lose moisture to the atmosphere, depending on the relative humidity and ambient temperature. The wood will eventually reach an equilibrium condition called equivalent moisture content (EMC) at which point the wood balances its moisture content with the moisture content of the surrounding environment. At EMC, there is no absorption or yielding of moisture from the wood. In most indoor coastal environments, EMC is between 10–15 percent. In dry inland areas, EMC can be as low as 6 or 7 percent.

12.13 Mechanical Properties of Lumber

Mechanical properties of lumber represent its ability to resist applied loads. Modulus of elasticity, compressive strength, modulus of rupture, tensile strength, and creep are the important mechanical properties of lumber that need to be considered for the structural design of lumber members. As discussed in Section 12.7, lumber is an anisotropic material; hence it has different properties in different directions. For structural design purposes, lumber can be considered to be an orthotropic material, which means that it has unique mechanical properties in the three mutually perpendicular axes. These mutually perpendicular axes are shown in Figure 12.10. The longitudinal axis is parallel to the grain, the tangential axis is perpendicular to the

Figure 12-10 Orthotropic nature of lumber



(Photo: Rabin Tuladhar)

grain and tangent to growth rings, and the radial axis is perpendicular to the grain and growth rings. The mechanical properties of lumber are significantly affected by moisture content, specific gravity, defects such as knots, and duration of loading. A decrease in moisture content improves the mechanical properties of lumber. However, below the fiber saturation point, a change in moisture content does not affect the mechanical properties of lumber. ASTM D198 and ASTM D143 specify the test methods for determining mechanical properties of structural-size lumber and small-clear lumber specimens, respectively.

12.13.1 Modulus of Elasticity

The modulus of elasticity (E) represents the stiffness of lumber, and it measures its resistance to deflection. Species type and moisture content in lumber affect its modulus of elasticity. For a given lumber member, the modulus of elasticity is also different along its longitudinal, radial, and tangential axes. The modulus of elasticity is highest in the longitudinal direction, and in the radial direction, it is about 10 percent of that in the longitudinal direction, and it is smallest in the tangential direction (about half of that in the radial direction).

12.13.2 Compressive Strength

The compressive strength of lumber significantly varies, depending on whether the force is applied perpendicular to the grain or parallel to the grain. The compression strength perpendicular to the grain is much smaller (around 10–20%) than that of parallel to grain. Compression strength parallel to the grain is used in the design of columns, posts, and compression members in trusses. The compression strength perpendicular to the grain is used to find the bearing strength of lumber beams.

12.13.3 Modulus of Rupture

The modulus of rupture is the bending strength of lumber. It represents the ultimate short-term load-carrying capacity of a beam when the load is applied slowly. The modulus of rupture is an important parameter used in the design of beams, joists, headers, and rafters. It varies with the species type and also depends on moisture content and specific gravity. Hardwood lumber is generally stronger in bending than softwood lumber. Bending members are also subjected to shear stresses. Shear failure in lumber is mainly caused by splitting due to the horizontal shear stresses along grain.

12.13.4 Tensile Strength

Lumber is strong in tension when force is applied along the grain. Tension members in trusses and studs in framed connection are subjected to tensile force along the grain. Tension failure is usually brittle and may cause catastrophic failure of structures. The tensile strength of lumber is greatly reduced by the presence of knots. However, variation in moisture content has relatively lower effect on tensile strength of lumber when compared to other mechanical properties. Tensile strength perpendicular to the grain is only around 5 percent of tensile strength parallel to the grain. It is further reduced by the presence of defects such as knots, cracks, and shakes.

12.13.5 Creep

Creep is the increase in deformation of a member subjected to constant load. When a load is applied to a lumber member, it is subjected to elastic deformation. If the load is sustained for a long time, additional creep deformation occurs. With the increase in moisture content, creep deformation in lumber members increases. Creep is also more significant when the load is applied perpendicular to the grain, rather than parallel to the grain. When the load is removed, elastic deformation is immediately restored; however, some creep deformation permanently remains as residual deformation.

12.13.6 Determination of Mechanical Properties

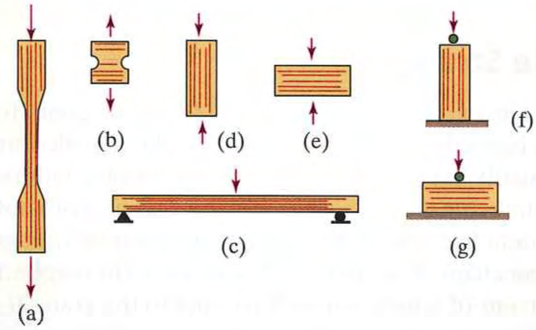
Testing on small, clear specimens obtained from the lumber structures is used to obtain and compare mechanical and physical properties of various lumber species and quality control of lumber production. Small-clear specimen testing of lumber is done according to ASTM D143 (2014). These tests include the static bending test, impact bending test, and compression and tensile tests parallel and perpendicular to the grain direction (Figure 12.11).

Mechanical properties can also be determined through the testing of structural-size lumber members, which are covered in ASTM 198 (2015). Properties obtained through structural-size tests are used in the design of structural members, development of grading rules and specifications, determining strength properties of different species in various structural sizes, and determining the effects of fabrication and joint types on structural properties of the members (ASTM D198, 2015).

12.14 Engineered Lumber Products

Apart from the traditional sawn lumber members, a wide range of engineered lumber products are used in the building and construction sector. These products are specially produced from wood-based components and are "engineered" to meet precise design specifications. Engineered lumber products are seasoned, dimensionally

Figure 12.11 Small-clear tests to determine mechanical properties (ASTM D143, 2014): (a) tension parallel to grain (b) tension perpendicular to grain (c) static bending (d) compression parallel to grain (e) compression perpendicular to grain (f) hardness parallel to grain (g) hardness perpendicular to grain



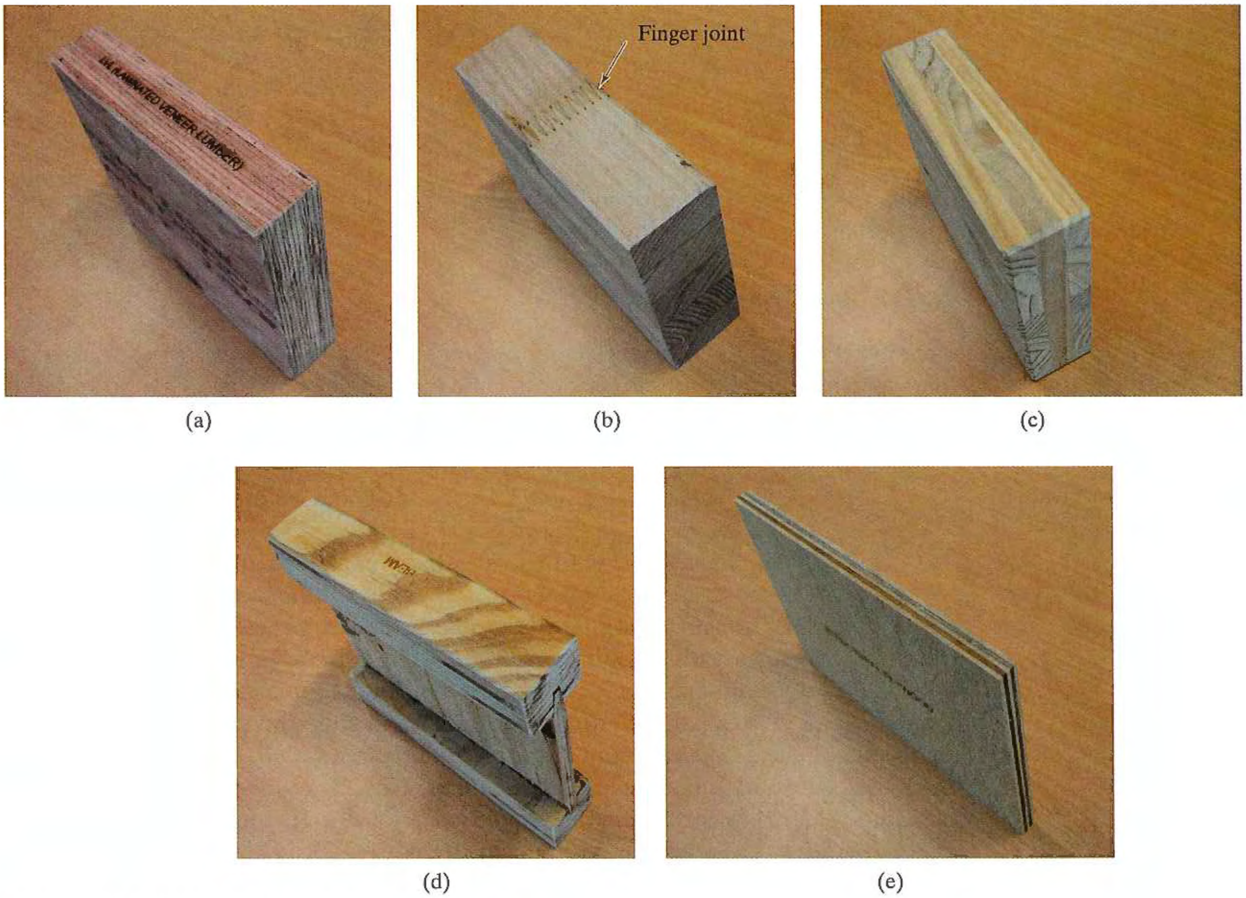
stable, uniform, and have a high strength-to-weight ratio. They can also be produced in long spans and curved profiles, making them a very attractive alternative to solid lumber members for special applications. Commonly used engineered lumber products in structural applications are laminated veneer lumber (LVL), glued laminated lumber (glulam), cross-laminated timber (CLT), I-beams, and plywood (Figure 12.12)

12.14.1 Laminated Veneer Lumber (LVL)

Laminated veneer lumber (LVL) is a high-strength engineered lumber produced by bonding together thin softwood veneers using waterproof adhesive (Figure 12.12a). Veneer is a thin sheet of wood cut from tree logs. In LVL, seasoned veneer layers are assembled, usually with grain parallel, and are glued together. It is then subjected to heat and pressure. LVL is stronger than natural lumber, is uniform, and does not contain natural defects. LVL is also less prone to shrinkage and warping.

Engineered lumber products (a) Laminated veneer lumber (LVL) (b) Glued laminated lumber (glulam) (c) Cross-laminated timber (CLT) (d) I-beam (e) Plywood

Figure 12.12



(Courtesy of R. Tuladhar, James Cook University, Australia)

Another big advantage of LVL is its ability to support heavier loads and it spans for longer distances. LVL products can easily be cut into different shapes. LVL is normally used in beams; hence, veneers are arranged with grain parallel to each other. Cross-banded LVL with laminations laid at right angles to enhance shear strength is also available.

12.14.2 Glued Laminated Lumber (Glulam)

Glued laminated lumber (glulam) is produced by bonding together layers of graded and seasoned lumber laminates (Figure 12.12b). Lumber laminates are arranged with grain parallel to each other. As glulam consists of layers of laminates glued together, defects are not present or are confined in only one laminate. It therefore has higher strength than the natural sawn lumber. Glulam can be produced in large sizes and long spans and can have different shape profiles as well. Lumber laminates in glulam can be finger jointed to produce continuous long products (Figure 12.12b). Glulam is reliable and dimensionally stable and is used widely in residential and commercial buildings as high-strength structural beams, industrial portal frames, and rafters. Glulam beams are often fabricated with camber to reduce deflection caused by dead loads.

12.14.3 Cross-Laminated Timber (CLT)

Cross-laminated timber (CLT) is an innovative engineered lumber product used for structural applications (Figure 12.12c). CLT is fabricated by bonding layers of lumber (known as lamellas) with the grain alternating at 90-degree angles in each layer. By having layers of lumber with alternating grain direction, the mechanical properties of CLT are greatly improved in both directions. CLT is less prone to shrinkage and swelling, and loads can be applied on CLT members in more than one direction. CLTs are used in floors, walls, ceilings, and roofs.

12.14.4 I-Beams

I-beams (Figure 12.12d) are I-shaped engineered lumber products mostly used as long-span structural beams in residential and commercial buildings. The top and bottom flanges of I-beams are made up of graded solid lumber or LVL. Structural plywood, oriented strand board (OSB) or hardboard is used as a vertical web. Flanges resist normal stresses caused by bending, whereas the web resists shear stresses. I-beams are structurally efficient, lightweight, and are available in long spans. I-beams are usually used for floor joists, roof purlins, and wall girts.

12.14.5 Plywood

Structural plywood is manufactured by assembling thin sheets of wood of uniform thickness (called veneers) and bonding them together to produce a flat sheet (Figure 12.12e). In plywood, veneers are assembled with alternate grain directions, which gives uniform strength in all directions. Both softwood and hardwood lumber can be used to produce plywood. Adhesive used in structural plywood is a type *A* phenolic resin, which is resistant to both weather and stress. Plywood is a versatile and lightweight engineered lumber product used for a wide range of interior and exterior applications.

12.15 Summary

1. Lumber is one of the oldest construction materials used by humankind. Wood harvested from forests and sawn to suitable dimensions for carpentry and building construction is called lumber.
2. Due to its relatively high strength-to-weight ratio, versatility, and constructability, lumber is extensively used worldwide to build residential houses, commercial buildings, bridges, and other structures.
3. Lumber is vulnerable to biological decay and termite attack. However, with proper preservative treatment, appropriate design and construction practice, careful site preparation, and regular maintenance, lumber decay and termite attack can be prevented in lumber structures.
4. Mechanical properties of lumber are significantly affected by moisture content, specific gravity, and species type.
5. Seasoning, or drying, of lumber members prevents shrinkage and improves dimensional stability, enhancing its mechanical properties. Seasoned lumbers are also less prone to decay and fungal attack.
6. Botanically, wood can be broadly classified as softwood and hardwood.
7. Softwood comes from evergreen coniferous trees, and hardwood comes from broadleaved trees. Softwoods are economical and generally easier to cut and work with than hardwoods. Hence, they are widely used in both residential and commercial constructions.
8. Hardwoods, on the other hand, are strong, durable, and robust. They are mostly used as high-strength structural members, bridge girders, and wharves.
9. Engineered lumber products, such as laminated veneer lumber (LVL), glued laminated lumber (glulam), plywood, and cross-laminated timber (CLT), further extend the versatility and structural application of lumber in the construction industry. These engineered products are seasoned, dimensionally stable, and uniform and can be produced in long spans and different profiles, making them a very attractive alternative to solid sawn lumber members.

Exercises

1. Heartwood is the:
   a. outer region of a tree trunk.
   b. center of a tree trunk.
   c. inner part of a tree trunk which gives structural support to a tree.
   d. outermost thin and rough layer of a tree trunk.

2. The number of growth rings in a tree approximately represents:
   a. diameter of the tree.
   b. age of the tree.
   c. strength of the tree.
   d. moisture content of the tree.

3. Choose the correct statement:
   a. Softwood comes from broadleaved trees.
   b. Softwood comes from angiosperm trees.
   c. Softwood comes from coniferous trees.
   d. Softwood refer to the softness of the wood.

4. A distinct feature of hardwood is:

   a. It has vessels running along the length of the tree.
   b. It is always hard.
   c. It is easy to cut and work with.
   d. It comes from coniferous trees.

5. Anisotropic materials are the materials with:

   a. similar properties along all the directions.
   b. different properties along different directions.
   c. high modulus of rupture.
   d. uniform cross-sectional area.

6. Seasoning is the process of:

   a. drying timber to equivalent moisture content (EMC).
   b. increasing moisture content above fiber saturation point.
   c. sawing timber into different shapes.
   d. harvesting trees from the forest.

7. Knots in timber can:

   a. decrease strength and stiffness of lumber.
   b. increase strength and stiffness of lumber.
   c. cause swelling of lumber.
   d. cause drying of lumber member.

8. The defect in lumber characterized by a curvature in the longitudinal direction of the plank is called:

   a. bow
   b. spring
   c. cup
   d. check

9. Glulam lumber is fabricated by:

   a. sawing timber logs radially so that growth rings are nearly perpendicular to the face.
   b. bonding layers of veneer.
   c. bonding layers of graded and seasoned laminates with grains parallel to each other.
   d. bonding layers of lamellas with the grains alternating at 90° angles in each layer.

10. An engineered lumber product manufactured by bonding thin veneers assembled with grain parallel is called:

   a. glued laminated lumber.
   b. cross-laminated timber.
   c. plywood.
   d. laminated veneer lumber.

11. Why is seasoning done on timber members? Explain different methods of seasoning.

12. What are the different types of defects that can exist in lumber members?

13. What are the main differences between softwood and hardwood? Give some examples of softwood and hardwood lumber species.

14. What are the main advantages of lumber compared to other construction materials like steel and concrete?

15. Do the research on various treatment methods that can be applied on lumber to protect it from biological decay and termite attack, enhancing its durability. Briefly explain those treatment methods.

16. What are the different mechanical properties of lumber that need to be considered in lumber design and construction? Discuss various parameters that effect the mechanical properties of lumber.

17. Explain shrinkage and creep in lumber structures.

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13 CHAPTER

Hearst Tower is one the most environmentally friendly office towers in New York City. Over 90% of its structural steel contains recycled material, and the innovative design eliminated the need for approximately 2,000 tons of steel. ben bryant/Shutterstock.com

Sustainability of Construction Materials

13.1 Introduction

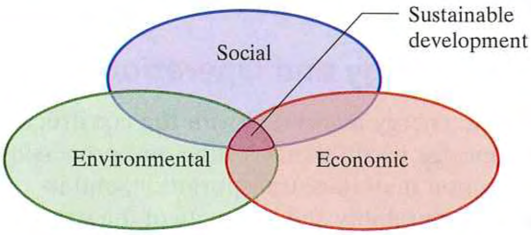
The construction and infrastructure industries play a vital role in the economic and social development of a society. But on the other hand, the construction sector is one of the biggest consumers of our natural resources, and it is also responsible for significant environmental impacts, including greenhouse gas emission (such as carbon dioxide, methane, and nitrous oxide), water and air pollution, and waste production. It is therefore very important for all the stakeholders in the building and construction industry to make responsible selection of building materials, construction techniques, and design methods and practices in order to optimize the use of natural resources and mitigate negative environmental effects. Careful selection of construction materials and their uses not only minimizes the resources and environmental effects, but it also yields significant improvements in the quality of life and cost effectiveness of our building environments. Any development that helps to enhance living standards by creating a safe and healthy living environment and improving social, economic, and environmental conditions for present and future generations is termed sustainable development. Stakeholders in the building and construction industry, from engineers and architects to suppliers and contractors, have the ability to influence the selection of materials and construction methods, and they should make conscious and informed decisions based on sustainability aspects of these materials, considering their entire life cycle.

13.2 Sustainable Development

Sustainable development is defined as development that meets the needs of the present without compromising the ability of future generations to meet their own needs (Brundtland, 1987). For an infrastructure to be sustainable, it has to serve its intended purpose, create a healthy and safe living environment, and be cost effective, while

Triple bottom-line framework for sustainable development

Figure 13.1



minimizing any adverse environmental impacts. Hence, sustainability of a product or process is to maintain a delicate balance of its economic, social, and environmental aspects, which are often referred to as the three pillars of sustainable development or the "triple bottom line" (Figure 13.1).

Traditionally, profit or the economy was the main driving force or the "bottom line" for businesses and industries. Understandably, any development initiatives need to be economically justifiable. The economic feasibility of a project is judged in terms of its whole-life-cost, economic output, and productivity. However, recently our society started looking beyond the mere monetary profits and has started giving importance on how the project will contribute to society at large and whether it will have any negative impacts on the environment. This gave rise to the triple bottom-line framework that evaluates the merits of any project, based on its economic, environmental, and social factors.

Social sustainability refers to the contribution of a project to improve quality of life and living standards of an individual or society in general. It also includes comfort, health, and safety of the occupants and the workers. Building and construction industry provides jobs and livelihood for millions of people, and hence it is an integral part of any community. Ensuring business practices that are fair to the community and the workers is also important to achieve social equity.

Environmental sustainability implies the preservation of air, water, land, and the ecosystem and minimizing potential harmful impacts on the environment, such as emission of greenhouse gasses, depletion of the ozone layer, acidification, air pollution, and eutrophication (contamination of water bodies with nutrients such as phosphates and nitrates). Furthermore, natural resources on our planet are limited. Irresponsible exploitation of our natural resources will aggravate depletion of resources. Using renewable energy, reducing consumption of nonrenewable natural resources such as fossil fuels, reusing and recycling of materials, and managing and reducing waste are examples of the various steps that can be taken to achieve environmental sustainability.

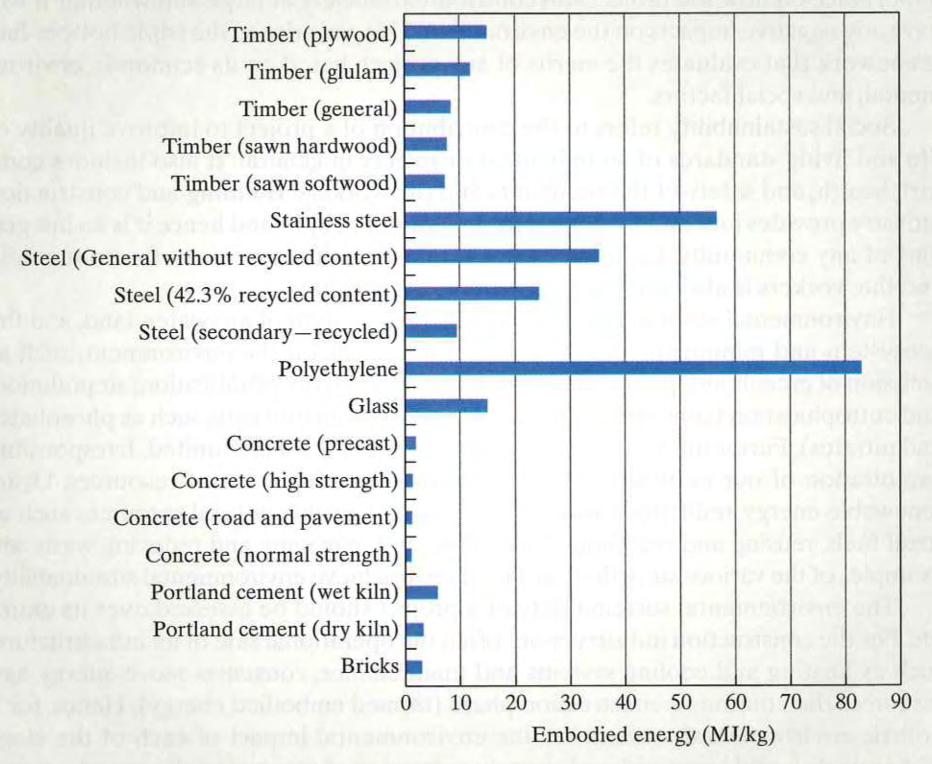
The environmental sustainability of a project should be assessed over its entire life. For the construction industry, more often the operational side of an infrastructure, such as heating and cooling systems and maintenance, consumes more energy and resources than during its construction phase (termed embodied energy). Hence, for a holistic environmental assessment, the environmental impact of each of the steps of a project should be considered: namely extraction of raw materials, manufacturing of construction materials, transportation, construction, operation and maintenance of the infrastructure, demolition, and waste management after its service life. Life-cycle assessment (LCA) offers a comprehensive and rigorous method to quantify all the environmental impacts associated with any product or services.

13.2.1 Embodied Energy and Operational Energy

Embodied energy is the energy associated with the construction of an infrastructure, which includes energy used for harvesting and processing of raw materials, production of construction materials, transportation, and its construction. Choice of construction materials, durability and longevity of the structure, and construction methods significantly influence the embodied energy of the structure. Embodied energy for commonly used construction materials is shown in Figure 13.2 (Hammond and Jones, 2008). It is, however, important to understand that embodied energy is only a component of total energy used by the infrastructure in its entire life and does not include energy required during the operational phase of the structure.

Operational energy is the energy required during the entire service life of the structure, such as lighting, heating and cooling systems, and maintenance. For example, for a typical detached house with a service life of fifty years, embodied energy can be as low as 10 percent of the total energy associated with the structure, the remaining

Figure 13.2 Embodied energy in different construction materials

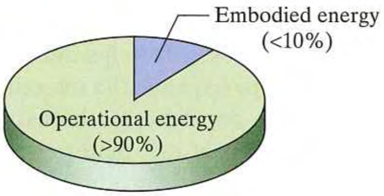


**Values for timber exclude carbon sequestration in timber (long-term storing of carbon).*

(Based on Hammond and Jones, 2008)

Embodied and operational energy of a typical house with a service life of fifty years

Figure 13.3



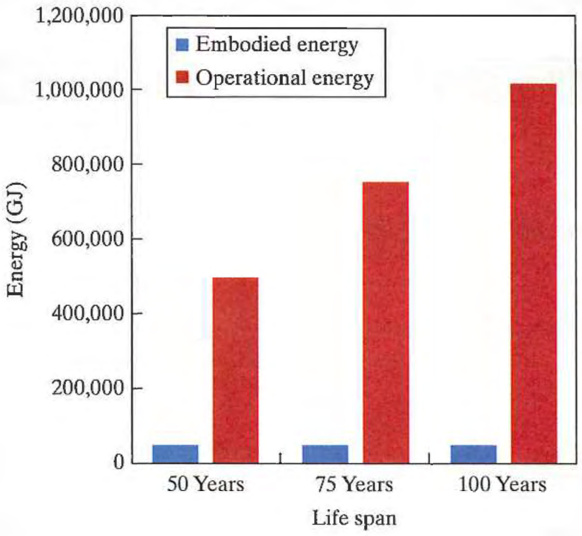
(Based on Cement concrete & Aggregate Australia)

90 percent being the operational energy (Figure 13.3). However, as energy-efficient buildings and appliances become more common, embodied energy will increasingly become a significant component of the total energy.

Operational energy can be reduced by implementing energy-efficient systems; however, the choice of construction materials can also make a significant saving in the operational energy as well. For example, the energy required for heating and cooling systems can be significantly reduced by using construction materials with high thermal mass or by using insulation. Embodied energy is incurred once, whereas operational energy accumulates over the service life of the structure. For an industrial building with a typical life span of 100 years, the difference between the embodied energy and operational energy is even more significant (Figure 13.4).

Embodied and operational energy of a typical commercial building for different life spans

Figure 13.4



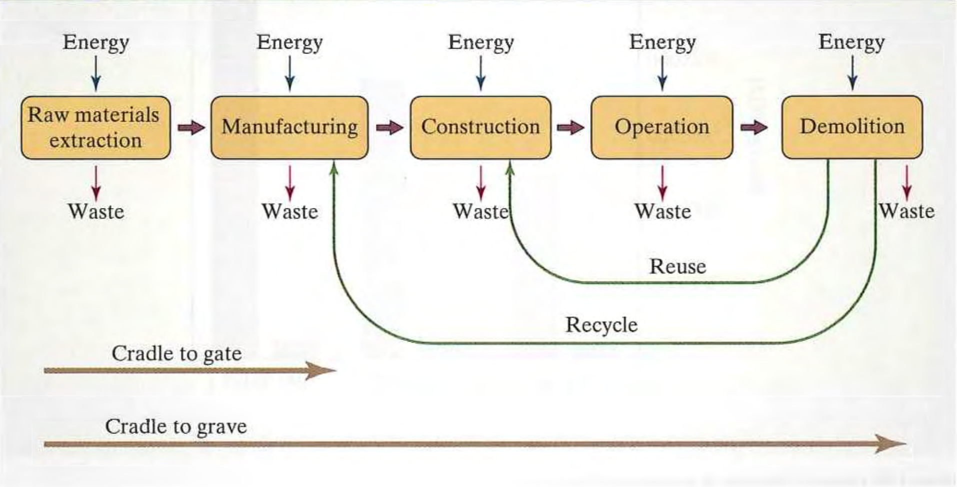
(Based on Cement concrete & Aggregate Australia)

13.2.2 Life-Cycle Assessment (LCA)

Life-cycle assessment is a holistic approach to systematically identify the resource flows and environmental impacts associated with a product, a service, or a process. LCA can provide a basis for comparing the environmental benefits or consequences of using different materials and processes. It helps decision makers to recommend sustainable materials and processes to improve the overall sustainability of the project. The life cycle of a product in a construction industry includes following five main stages: (a) mining and extraction of raw materials; (b) processing and manufacturing of construction materials (such as production of cement, concrete, steel, timber, aggregates, bitumen, masonry); (c) construction of the structure, including both prefabrication and on-site construction; (d) operation of the structure (which includes lighting, heating, cooling systems); and (e) demolition of the structure after it serves its purpose. Each of these steps consumes energy and has environmental consequences such as waste production, greenhouse gas emissions, and so forth (Figure 13.5). After a structure reaches its service life, it is demolished. Components from the construction and demolition waste can be reused in some other forms, or the materials may be recycled back into the system. The remaining waste goes into the landfill or is incinerated. A complete life-cycle assessment should take into account energy and emissions associated with a product or services throughout its entire life cycle from raw material acquisition to production, use, transportation, operation, end-of-life treatment, recycling, and final disposal (Figure 13.5). This comprehensive approach of doing LCA for the whole of life of a product is termed "cradle-to-grave." Sometimes LCA only limits its scope to a certain stages of a product life, for example, from extraction of a material to manufacturing of a product. This approach is called "cradle-to-gate" and only accounts for the environmental impacts of specific stages of the product's life.

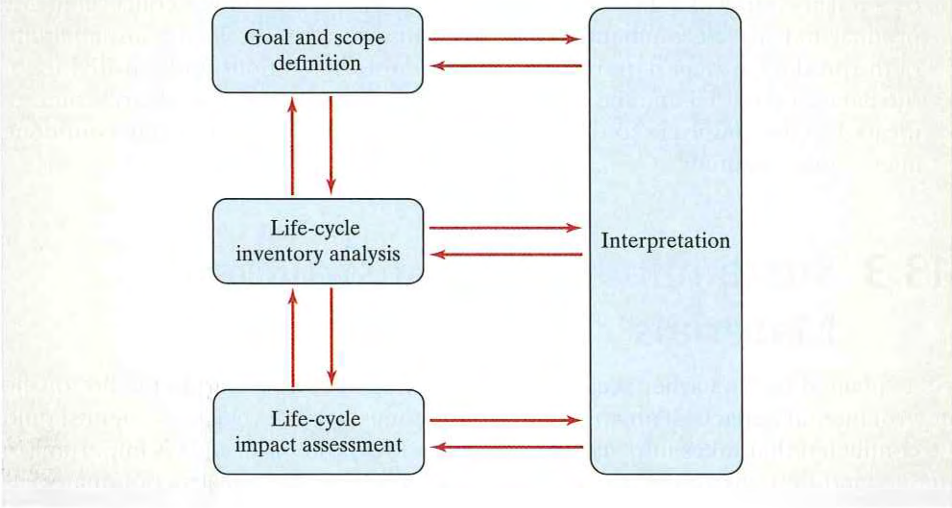
ISO14040 (2006) and ISO14044 (2006) provide a framework for LCA to compile, evaluate, and analyze the inputs and outputs, and the potential environmental impacts of a system throughout its life cycle. According to this international standard, LCA comprises four distinct phases: goal and scope definition, life-cycle inventory (LCI), life-cycle impact assessment (LCIA), and interpretation.

Figure 13.5 Life-cycle assessment, considering whole life-cycle phases



Life-cycle assessment framework

Figure 13.6



(Based on ISO 14040, 2006)

a. ***Goal and scope definition:*** The very first step of an LCA is to explicitly define the context and scope of the assessment and identify the system boundaries and environmental effects to be considered in the assessment. LCA may take the cradle-to-grave approach, where the system boundary includes entire life of a product, or a cradle-to-gate approach, where only specific stages in a life of a product is reviewed. It is also important to identify a functional unit for the study, which defines what is exactly being studied and gives a fair ground for comparison between two alternative products. For example, a functional unit of a study can be the comparison between the use of steel mesh or plastic fibers to achieve similar reinforcing effects for a 1000 m^2 (10,760 ft^2) concrete footpath. Goals and scope also define who will be the end users of the LCA results.

b. ***Life-cycle inventory:*** The life-cycle inventory (LCI) identifies different life-cycle stages of a product within the scope of the study and compiles a database on resources inputs (such as energy, water, land, etc.) and waste outputs (such as atmospheric emissions, waterborne emissions, solid wastes, etc.) associated with each of the life-cycle stages. Flow charts are used to illustrate different life-cycle stages and their inputs and outputs. Sources from where the relevant data are collected should clearly be defined in the LCI as well.

c. ***Life-cycle impact assessment (LCIA):*** A life-cycle impact assessment (LCIA) assesses and quantifies the potential human and ecological effects and resource depletion due to the resource used and environmental releases at different stages of a product as identified in the LCI. Various impact categories such as global warming, ozone depletion, eutrophication, water and land usage, and human toxicity can be used to characterize and quantify the actual effects on human beings and ecosystem. For example, say LCI identifies that a process emits 1,000 tons of carbon dioxide and 3,000 tons of methane into the atmosphere. LCIA will quantify the actual impacts of these releases on the ecosystem, using indicators like global warming and ozone layer depletion.

d. ***Interpretation:*** Interpretation of an LCA is an important step to evaluate and summarize overall environmental impacts of a process or to compare multiple processes. This step will help to flag significant environmental issues concerning any product and gives recommendations on various ways to improve the sustainability of the product. As a part of the LCA, uncertainties, limitations, and sensitivities of the data used in LCI and the results thus obtained should also be clearly communicated to the end users so that they can interpret the results in a fair, confident, and accurate manner.

13.3 Sustainability of Construction Materials

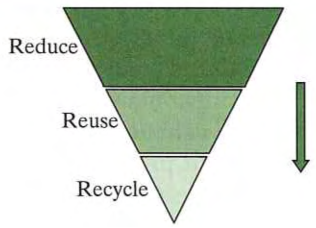
As explained in the earlier sections, to get a complete and accurate picture of the environmental impacts of any project, a comprehensive life-cycle assessment should be conducted that takes into account the entire life of the project. It is important to understand that environmental impacts associated with the construction materials used in the project form only a part of the overall impact of the project. Nevertheless, construction materials are core components of any construction project, and it is very important for the stakeholders to understand the sustainability performance of construction materials and make a conscious choice of materials that will improve the sustainability of the entire project. With the increase in focus on using energy-efficient heating and lighting systems in a building, the contribution from embodied energy of construction materials is becoming increasingly more significant. Furthermore, with the increasing concerns of resource depletion, engineers, specifiers, designers, and contractors are constantly facing challenges to develop and use alternative sustainable materials.

The most important consideration toward achieving sustainable development is to reduce the demand and use of new (virgin) materials. Innovative design methods can reduce the total consumption of construction materials. For example, the use of high-strength concrete and prestressed concrete technologies can significantly reduce the thickness of concrete slabs and girders, reducing the overall materials used in the project. Instead of demolishing a structure, consideration should also be made to reuse the space in different forms. Designers should be encouraged to use innovative techniques that enable easy dismantling and reuse of building components. Steel structures and prefabricated concrete structures can be dismantled easily to reuse their structural components in other applications. Furthermore, construction and demolition waste can also be recycled to reduce the use of virgin materials. Use of durable construction materials will reduce maintenance and increase the longevity of the structure, which will contribute toward the overall sustainability of the structure. As shown in Figure 13.7, the sustainability hierarchy encourages, first and foremost, to reduce consumption of natural resources wherever possible, then to reuse the material without much processing, and finally, to recycle the waste.

Another important step toward sustainable development is to ensure the use of construction materials with low carbon footprints. The carbon footprint measures the potential contribution of human activities on climate change as expressed in terms of the weight of CO_2 equivalent. Sustainability performance of construction materials can be made by comparing various available materials, using LCA. Use of sustainable materials not only will reduce the net negative environmental impact of

Sustainability hierarchy: Reduce, Reuse, and Recycle

Figure 13.7



the project, but also will make it cost effective, improve its productivity, and reduce waste. The following sections covers intrinsic sustainability benefits and limitations of most commonly used construction materials.

13.3.1 Cement and Concrete

Concrete is the second most used material by humankind, second just to water. The data shows, as of 2012, that 25 billion tons of concrete are used globally every year (Schokker, 2012). Due to the sheer volume of concrete poured around the world, it is very important to understand the sustainability performance of concrete. Concrete offers some distinct social and economic sustainability benefits. Raw materials required for producing concrete, such as limestone, water, sand, and aggregates, are cheap and easily available around the world, making concrete one of the cheapest construction materials. Furthermore, concrete structures are very durable and require minimum maintenance and treatment. Steel and timber require special treatments and regular maintenance, such as corrosion resistance painting for steel and treatments against termite attacks and biological degradation for wood. Furthermore, concrete has high thermal mass (ability to store energy within its mass); hence, concrete structures are energy efficient and consume less energy for heating and cooling systems. Long-term operation and maintenance costs are also low for concrete structures. Concrete construction techniques, such as tilt-up panel construction and precast and prestressed concrete technologies, are some of the most efficient construction methods, often providing the most economic structural solutions. Concrete in construction and demolition waste can also be recycled as road subbase materials or aggregates for concrete.

On the social front, concrete provides a safe and healthy living environment. The bulkiness of concrete members leads to low structural vibration. Vibrations like wind-induced vibrations are much smaller in concrete structures compared to more slender steel construction. If constructed properly, concrete structures also provide excellent resistance against weather and natural disasters such as flooding, earthquakes, and cyclones. Concrete is an inert and nontoxic material and has the best fire resistance performance compared to other building materials. As seen in Figure 8.2, concrete is often used to encase steel members to protect them from fire damage. Other distinct advantages of concrete are termite resistance, blast and impact resistance, and excellent acoustic performance.

Sand, coarse aggregates, and water constitute almost 90 percent of concrete. Embodied energy associated with these materials is minimal. However, the manufacturing of cement, a chief constituent of concrete, is a very energy-intensive process. Around 85 percent of the total embodied energy of concrete is attributed to cement. As explained in Section 8.2.1, the cement manufacturing process consists of extracting raw materials, burning raw materials in a rotary kiln at a very high temperature (around 1500°C) to form clinkers, and crushing the clinkers to form cement. CO_2 emission associated with the cement manufacturing process mainly comes from burning fossil fuels in kilns and the calcination of limestone that goes inside the kiln (Section 8.2.1). It is estimated that production of 1 tonne of cement produces approximately 1 tonne of carbon dioxide. Cement industries account for five percent of all human CO_2 emission globally (WBCSD, 2012). Hence, from an environmental standpoint, CO_2 emission is an important sustainability concern for the cement and concrete industries around the world. The concrete and cement industries have been taking various measures to reduce the environmental impact of cement and concrete production.

Instead of relying on fossil fuel, nontraditional fuels such as used tires, oils, and waste timber are increasingly being used by the cement industry to burn raw materials in the kilns. Use of alternative fuels, which are usually byproducts of other manufacturing processes, lead to both environmental and economic benefits.

Use of supplementary cementitious materials (SCMs) to partially replace portland cement in concrete also reduces the consumption and production of cement (Section 8.2.2). Fly ash (byproduct of coal-fired thermal power plants), ground granulated blast furnace slag (byproduct of iron industries), and amorphous silica (naturally occurring materials and byproduct of the silicon and ferrosilicon industries) are the commonly used SCMs in concrete. Use of SCMs in concrete as a partial replacement of cement has a twofold advantage: it reduces the carbon footprint of concrete by reducing cement consumption, and it recycles industrial waste that would otherwise end up in landfills. The use of other innovative materials, such as geopolymer concrete, is also gaining traction in the construction industries. Geopolymer concrete is produced by reacting alumina silicate materials (such as fly ash or slag) with a caustic activator (such as sodium hydroxide, potassium hydroxide, sodium silicate, and potassium silicate). Use of a geopolymer binder can entirely replace portland cement in concrete contributing to significant environmental benefits.

Coarse aggregate and fine aggregates (sand) in concrete are naturally occurring materials and need minimal energy and cost to extract. Natural sand is, however, depleting in many parts of the world, due to its excessive extraction. A significant portion of natural sand in concrete can be partially replaced using manufactured sand. Manufactured sand is a byproduct of crushing stones in quarries to produce coarse aggregates. Its use in concrete reduces the extraction of natural sand and also helps to reuse quarry waste efficiently (Figure 13.8). Apart from manufactured sand, recycled aggregate from construction and demolition waste can also be used as coarse aggregate for producing low-strength concrete. Most of the concrete plants now recycle wash water collected from washing concrete trucks and water run-off from concrete plants. Concrete wash water is highly alkaline and should not be discharged directly into waterways, as it is very detrimental to aquatic life. Recycling wash water into a concrete mix helps prevent contamination of waterways and rivers, and at the same time, will reduce demand on municipal water supplies.

Various other innovative research in the concrete industry is looking into improving its environmental sustainability. Researchers have shown that recycled plastic fibers can be used to replace steel mesh to reinforce concrete footpaths and precast elements, such as drainage pits (Figure 13.9a). Steel mesh is traditionally used in

(a) Manufactured sand obtained as a byproduct in quarries while crushing stones can be used as partial replacement of natural sand in concrete; (b) example of concrete made using manufactured sand as a partial replacement of natural sand

Figure 13.8


(a) (b)

(Courtesy of R. Tuladhar, James Cook University, Australia)

(a) Concrete footpath reinforced with recycled plastic fiber at James Cook University, Townsville, Australia; (b) recycled plastic fiber and its use in concrete

Figure 13.9


(a) (b)

(Courtesy of R. Tuladhar, James Cook University, Australia)

concrete footpaths to control plastic and drying shrinkage in concrete. Replacement of steel mesh with recycled plastic fibers (Figure 13.9b) helps reduce the use of steel in concrete by replacing it with recycled plastic waste (Yin et al., 2015).

Photocatalytic concrete is another novel technology recently developed that can reduce air pollution. Chemicals such as titanium dioxide (TiO_2) can be mixed with concrete to produce photocatalytic concrete, which has ability to absorb air pollutants (nitrogen oxides, NO_x and sulfur oxides, SO_x) and break them into nonharmful elements in the presence of sunlight (Schokker, 2012).

13.3.2 Steel

As discussed in Section 10.1, 1.6 billion tonnes of steel are manufactured every year, out of which more than 50 percent is used in the construction industry. With its versatility, high strength-to-weight ratio, cost effectiveness, and ductility (ability to deform or shaped without breaking), steel is one of the most preferred construction materials and is an integral part of modern civilization. The most unique advantage of steel is its sustainability compared to other construction materials, because it is 100 percent recyclable. Steel can be recycled repeatedly without any degradation in its mechanical properties and performance. As steel is manufactured in a controlled industrial environment and is efficiently fabricated at site, steel construction leads to very little or no waste. Furthermore, steel structures can easily be dismantled and reused again. Reuse of building components offers even greater sustainability advantage than recycling. Steel's high strength-to-weight ratio results in slender members and smaller foundations, which eventually means less use of materials, less transportation, and less energy and emissions. Erection of steel structures at site is also less labor intensive compared to other conventional construction techniques.

Steel industries have continuously been striving to develop technologies to reduce the amount of energy required to produce steel. Embodied energy per kilogram of material for steel is higher than other construction materials like concrete or timber (Figure 13.2). Production of primary steel from iron ore in a blast furnace, as described in Section 10.4.1, is an energy-intensive process requiring high temperature and water. The ore reduction process, the chemical reaction between coal, carbon, and iron ore in a blast furnace, generates the majority of CO_2 associated with steel production. Modern steel-making processes feed all the generated hot gases back into the system, thus reducing the power and heat requirement of the system and reducing energy requirements. Basic oxygen steelmaking (BOS) these days typically uses 25–35 percent of steel scrap along with molten iron ore. Use of scrap steel helps lower its embodied energy. An electric arc furnace (EAF) is a more resource- and energy-efficient method of steel production, as it uses 100 percent scrap metal and uses less energy as well (Section 10.4.2). Secondary steel produced from recycled steel scrap has much lower embodied energy (Figure 13.2).

Corrosion is the biggest threat for steel structures, which can cause loss of strength. Protective coating/painting and regular maintenance are required in steel structures. Steel properties can also degrade drastically at high temperature or fire. Hence, preventive measures like encasing steel members with concrete should be done to protect steel members from fire damage. The regular maintenance requirement can increase operational cost and environmental impact for steel structures. Proper design, construction techniques, and regular maintenance will reduce the operational cost of steel structures, ensuring clean, efficient, and sustainable infrastructures.

13.3.3 Wood

Wood is a natural material. It is easier and less energy intensive to process wood compared to other construction materials. Embodied energy required for manufacturing lumber is shown in Figure 13.2. It should be noted that embodied energy for lumber mentioned in Figure 13.2 does not include its carbon sequestrating property (absorbing and storing of carbon) and its calorific value (energy released when used lumber is burnt as a fuel). As a tree grows, it absorbs carbon dioxide and releases oxygen to the atmosphere. The carbon dioxide thus observed is stored as biogenic carbon in the wood tissues even when it is transformed into lumber products. Carbon will stay trapped in lumber until it is burned or gets rotten. This process of absorbing and storing of carbon for a very long time is called carbon sequestration. Hence, well-managed tree plantations for lumber products help decrease CO_2 from the atmosphere. Additionally, use of lumber in construction ensures that carbon remains locked in the lumber products, which is often more than the carbon released during their production. Used lumber can also be easily recycled as construction materials, in horticulture industries, or as a fuel. Lumber is a biodegradable material; hence, it does not create harmful pollution like other kinds of construction and demolition wastes. The amount of CO_2 released by burning or biological degradation of lumber is similar to the amount of CO_2 absorbed during the growth of lumber, hence making lumber a carbon neutral material.

Lumber structures are also lightweight and with the advent of engineered lumber products, like laminated veneer lumber (LVL), glued laminated wood (glulam), and cross-laminated timber (CLT), lumber can be efficiently used to build large structures. Lumber also has a very good insulating property, which improves thermal efficiency in houses. Moreover, additional insulation material can be placed in spaces between lumber frames. Lumber naturally has good acoustic properties. Hence, it is also used as claddings, wall linings, and ceilings in theaters, auditoriums, and public buildings to reduce undesirable echoes and reverberations.

Lumber, however, requires regular treatment and application of preservatives to prevent termite attacks and degradation from weather and fungi. Regular maintenance adds to operational costs for lumber structures. Nevertheless, with appropriate design and construction practice, careful site preparation, and regular maintenance and inspection, lumber structures are durable and can last for a very long time. Seasoning is also done on lumber members to reduce moisture content in harvested lumber. Air seasoning consumes minimal energy; however, energy is required if a kiln drying process is used. Most of the modern kilns use waste from the wood industry as a fuel-saving fossil fuel.

It should be emphasized that timber should be harvested responsibly from managed plantations only. It is very important to note that haphazard timber harvesting and logging create serious deforestation and ecological disasters. Deforestation further causes soil erosion and the degradation of biodiversity. Timber should only be harvested from sustainably managed forests where new plantings are regularly done to replace felled trees.

13.3.4 Soils, Rocks, and Aggregates

Geomaterials, which include soils, rocks, and aggregates, are naturally occurring and abundantly available materials. Soils and rocks are used as construction materials in earth- and rock-fill dams, road work, embankments, backfills for retaining walls, and land reclamation. Aggregates are basically derived from rocks and are classified as natural materials (such as sand and gravel) and manufactured materials (such as fine and coarse aggregate obtained from crushing larger stone). Aggregates are used in large quantities in concrete, asphalt, mortar, road sub-bases, and railway ballast. Extraction and processing of these geomaterials are usually low-cost and consume minimal energy. Transportation of these materials from distant sources is usually the biggest component of the cost and energy associated with these materials. However, irresponsible exploitation of soils, rocks, and aggregates can cause depletion of natural resources and lead to secondary effects, such as soil erosion, dust emission, and deforestation.

With increasing concern about sustainability, alternative materials, such as construction and demolition wastes, and industrial byproducts like fly ash and slag, have started to gain attention as substitutes for soils and aggregates in construction industries. As explained in Section 3.2, small doses of lime or supplementary cementitious materials like fly ash, silica fume, and slag are sometimes used to improve soil. Supplementary cementitious materials are also used as partial replacement of portland cement in mining backfills, along with crushed waste rocks and mine tailings. Use of dredged mud in land reclamation and as engineered fill materials addresses environmental concerns of dumping dredged spoil in the sea.

Recycled aggregates obtained from crushing construction and demolition waste (Figure 13.10) can be used as subbase materials in construction of roads and pavements. Attempts have been made to use recycled aggregates in concrete, as well, with limited success. Use of recycled aggregates reduces consumption of virgin aggregates and demolition waste going into landfills. However, it is important to understand the performance characteristics of recycled aggregate compared to the natural aggregates before using it in construction. Recycled aggregates should meet stringent requirements of minimum physical and plastic properties before they can be used in road subbases. Use of recycled aggregate from demolition waste in concrete can negatively affect its strength, workability, shrinkage, and creep behavior. The percentage of replacement of natural aggregate with recycled aggregate in concrete should be carefully calculated, based on mix design, properties, and the source of recycled aggregate and its availability. Crushing and screening of construction and demolition waste can also be costly and energy intensive. The energy and cost required for crushing and screening of construction and demolition waste should also be factored in when specifying the use of recycled aggregates in construction.

13.3.5 Asphalt Concrete

Asphalt concrete is produced by mixing asphalt and aggregates. Asphalt, also known as bitumen, is primarily derived from crude oil. Asphalt is the black, sticky, viscous thermoplastic residue obtained after distilling out light- and medium-weight hydrocarbons from crude oil. Asphalt concrete formed by mixing aggregates and asphalt is widely used as paving material for road pavements, roofing, waterproofing, and other

Figure 13.10 Construction and demolition waste that was used to produce subbase material for the new building at the site

(Courtesy of R. Tuladhar, James Cook University, Australia)

industrial applications. Asphalt concrete is usually applied at a temperature range of 140 °C to 160 °C, at which it is softer and fluid, making it easier to apply. Asphalt concrete also provides skid resistance and a low-noise road surface for comfortable driving. Asphalt pavements can also be easily repaired without causing much traffic disruption.

Asphalt concrete contains only a small percentage (3–7% by weight) of asphalt as a binder material, and aggregates form the bulk of its volume. Due to the low embodied energy of aggregates, the total embodied energy associated asphalt concrete is significantly lower compared to the asphalt itself. Modern technologies available for distilling crude oil to produce asphalt result in the low emission of hydrocarbons (hydrocarbons are carbon and hydrogen compounds, which contribute to greenhouse effect, global warming, water pollution, and damage to ecosystems). Crude oil is a natural resource, and as with every natural resource, the availability of crude oil in the nature is finite. Asphalt, however, can be 100 percent recycled without degrading its quality and properties. It is one of the most recycled construction material in the world (EAPA, 2014). Recycled asphalt product (RAP) is the pavement material removed from road surfaces. RAP chunks are crushed, screened, and heated at an appropriate temperature with virgin materials, and the hot mix, thus formed, can again be reused as asphalt concrete. Further development and innovation in asphalt production, recycling methods of RAP and regular maintenance to improve service life and recyclability of asphalt will further improve its sustainability.

13.4 Summary

1. The sustainability of a product or a process is to maintain a delicate balance of its economic, social, and environmental aspects, which are referred to as the triple bottom line.
2. Conscious and careful selection of construction materials will minimize the depletion of natural resources and minimize negative environmental impacts. It also helps to make infrastructures cost effective and create safe and healthy living environments for the occupants.
3. The operational phase of infrastructure often consumes more energy and resources than the construction phase.
4. The sustainability of a project should be assessed, based on the performance of the structure in its entire life. A life-cycle assessment (LCA) is a holistic tool to quantify all the environmental impacts associated with all the phases of the product, including extraction of raw materials, processing, transportation, construction, operation, and demolition.
5. An LCA provides a basis for comparing the environmental benefits or consequences of using different construction materials and processes, helping decision makers to make conscious choice of materials.
6. The most important consideration toward achieving sustainable development is to reduce the use of virgin materials and to maximize reusing and recycling materials.
7. The use of sustainable materials and construction processes not only will reduce the net negative environmental impact of the project but also will make it cost effective, improve its productivity, and reduce waste.

Exercises

1. Use of sustainable construction materials and methods make infrastructures:
   - **a.** cost effective
   - **b.** environmental friendly
   - **c.** socially responsible
   - **d.** all of the above
2. Embodied energy is energy associated with the:
   - **a.** operation of the infrastructure
   - **b.** demolition of the infrastructure
   - **c.** harvesting, processing of raw materials, and production of construction materials
   - **d.** none of the above
3. Choose the correct statement:
   - **a.** Embodied energy is only a component of the total energy used by the infrastructure in its entire life.
   - **b.** Embodied energy includes energy required during the operation phase of the structure.
   - **c.** Operational energy is associated with the manufacture of the product.
   - **d.** Embodied energy is the only consideration required to achieve sustainability of the product or process.
4. Choose the correct statement:
   - **a.** The operational energy of an infrastructure can be reduced by implementing energy-efficient systems.
   - **b.** The total operational energy for an infrastructure remains constant throughout its service life.
   - **c.** The operational energy is only a small component of the total energy used by the infrastructure in its entire life.
   - **d.** The operational energy of an infrastructure is usually much smaller than its embodied energy.
5. Cradle-to-grave life-cycle assessment methodology takes into account environmental impacts of the following phases:
   - **a.** extraction and processing of raw materials and construction phase
   - **b.** operation of the structure
   - **c.** demolition of the structure
   - **d.** all of the above
6. The embodied energy of concrete can mainly be attributed to:
   - **a.** aggregate
   - **b.** water
   - **c.** cement
   - **d.** admixtures
7. CO_2 emission associated with cement manufacturing process mainly comes from:
   - **a.** burning of fossil fuel and calcination of limestone
   - **b.** additional of gypsum
   - **c.** grinding of cement
   - **d.** storage of cement

8. The use of supplementary cementitious material in concrete will:
   - a. reduce the carbon footprint of concrete
   - b. increase waste going into landfill
   - c. decrease the strength of concrete
   - d. make concrete expensive
9. Choose the correct statement:
   - a. Steel is 100 percent recyclable.
   - b. The basic oxygen steel-making process is more efficient than the electric arc furnace (EAF) process.
   - c. Scrap metal cannot be used in the electric arc furnace (EAF) process.
   - d. Steel cannot be recycled.
10. Carbon sequestration is a process of:
    - a. releasing carbon dioxide into atmosphere
    - b. absorbing and storing carbon for a long period of time
    - c. releasing oxygen into atmosphere
    - d. dissolving carbon dioxide
11. What do you mean by a triple bottom line? Why is it important to consider the triple bottom line for sustainable development?
12. What are the different steps in life-cycle assessment? Describe how life-cycle assessment can help in sustainable development.
13. What are the main differences between cradle-to-gate and cradle-to-grave approaches in life-cycle assessment methodology?
14. What are the intrinsic sustainability benefits of concrete? Are there any environmental challenges concerning concrete?
15. Explain different sustainability measures that can be adopted in the use of geomaterials.

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APPENDIX A: UNIT CONVERSIONS

Length:

- angstrom = 10^{-10} m
- foot (ft) = 0.3048 m
- inch = 2.5400×10^{-2} m
- mil = 1/1000 inch = 2.54×10^{-5} m
- mile = 1.609344 km
- nautical mile = 1852 m
- yard = 3.00 ft = 0.9144 m

Area:

- acre = 4840 sq. yard = 4046.8564224 m^2
- hectare (ha) = 10,000 m^2
- perch = 25.29285246 m^2
- square = 9.290304 m^2

Volume:

- gallon (U.S.) = 3.785412×10^{-3} m^3
- gallon (U.K.) = 4.546090×10^{-3} m^3
- litre = 1.0×10^{-3} m^3
- ounce (U.S. fluid) = 2.957353×10^{-5} m^3
- pint (U.S. fluid) = 4.731765×10^{-4} m^3

Mass:

- British ton (short) = 2000 lb
- British ton (long) = 2240 lb = 20 hundredweight
- hundredweight (U.K.) = 112 lb
- hundredweight (U.S.) = 100 lb
- metric ton (or tonne) = 1000 kg
- ounce (mass) = 28.34952 g
- pound (mass) = 0.4535924 kg
- U.S. ton = 2000 lb

Force:

- dyne (dyn) = 1.00×10^{-5} N
- kilogram-force = 1.00 kilopond = 9.80665 N
- kip-force (1000 lbf) = 4448.222 kN
- poundal = 0.13825495 N
- pound force (lbf) = 4.448222 N

Pressure:

- atmosphere (760 mm Hg) = 1.013247×10^5 Pa
- bar = 1.0×10^5 Pa
- kgf/cm^2 = 98.06650 kPa
- lbf/in^2 (psi) = 6.894757 kPa
- kips/in^2 (ksi) = 6.894757 MPa
- ton-force (short)/sq.ft = 95.760518 kPa
- ton-force (long)/sq.ft = 107.251780 kPa

Work or Energy:

- Btu = 1055.0559 J
- calorie = 4.186800 J
- erg = 1.00×10^{-7} J
- joule (J) = 1.000 N·m
- kilowatthour = 3600 kJ

Dynamic viscosity:

- poise (P) = 0.1 Pa·s
- centi poise (cP) = 0.001 Pa·s

Kinematic viscosity (dynamic viscosity divided by density):

- Stoke (St) = 1.0 cm^2/s = 1.0×10^{-4} m^2/s
- Centistoke (cSt) = 1.0×10^{-6} m^2/s

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