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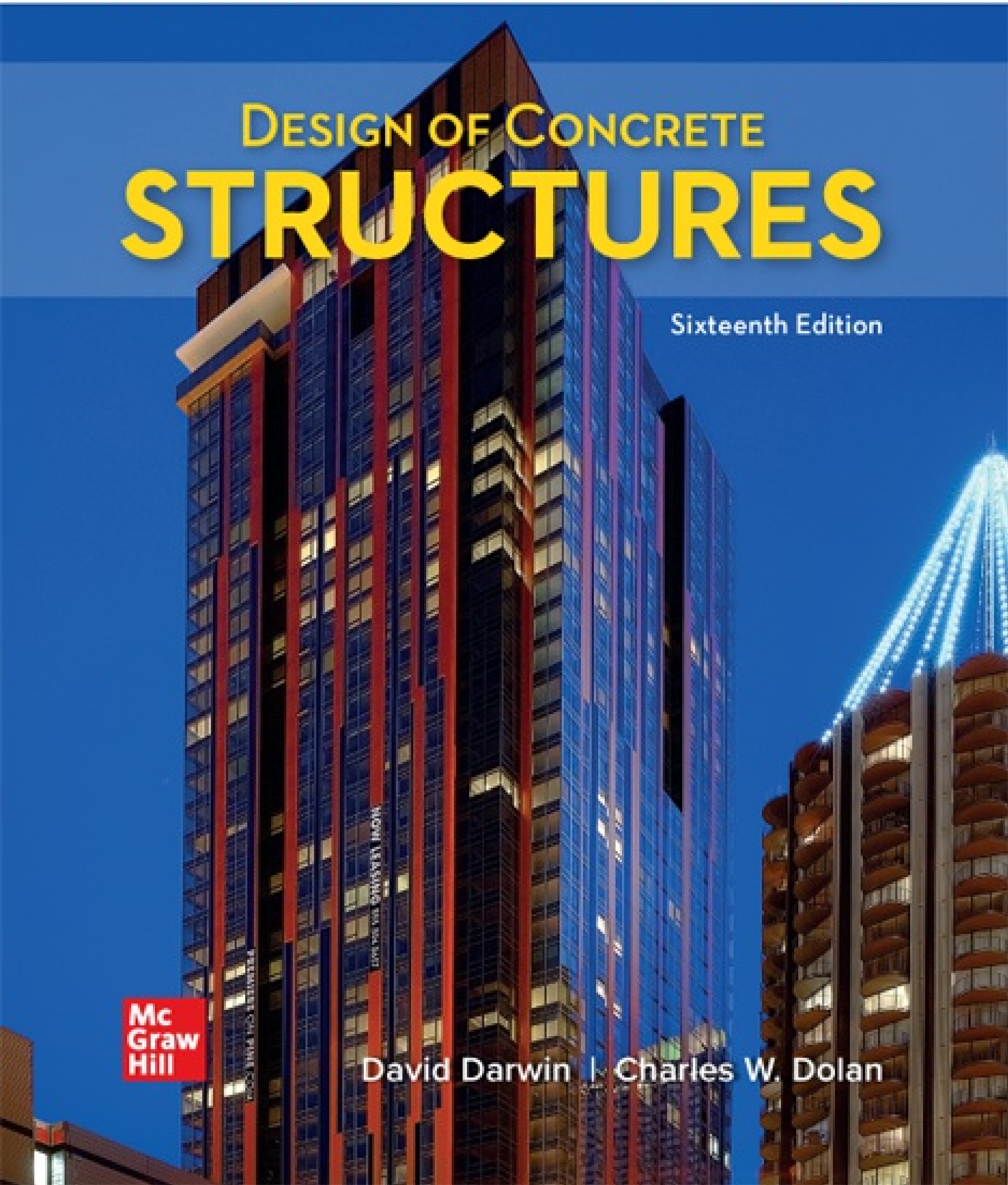

Design of Concrete Structures

Sixteenth Edition

David Darwin | Charles W. Dolan

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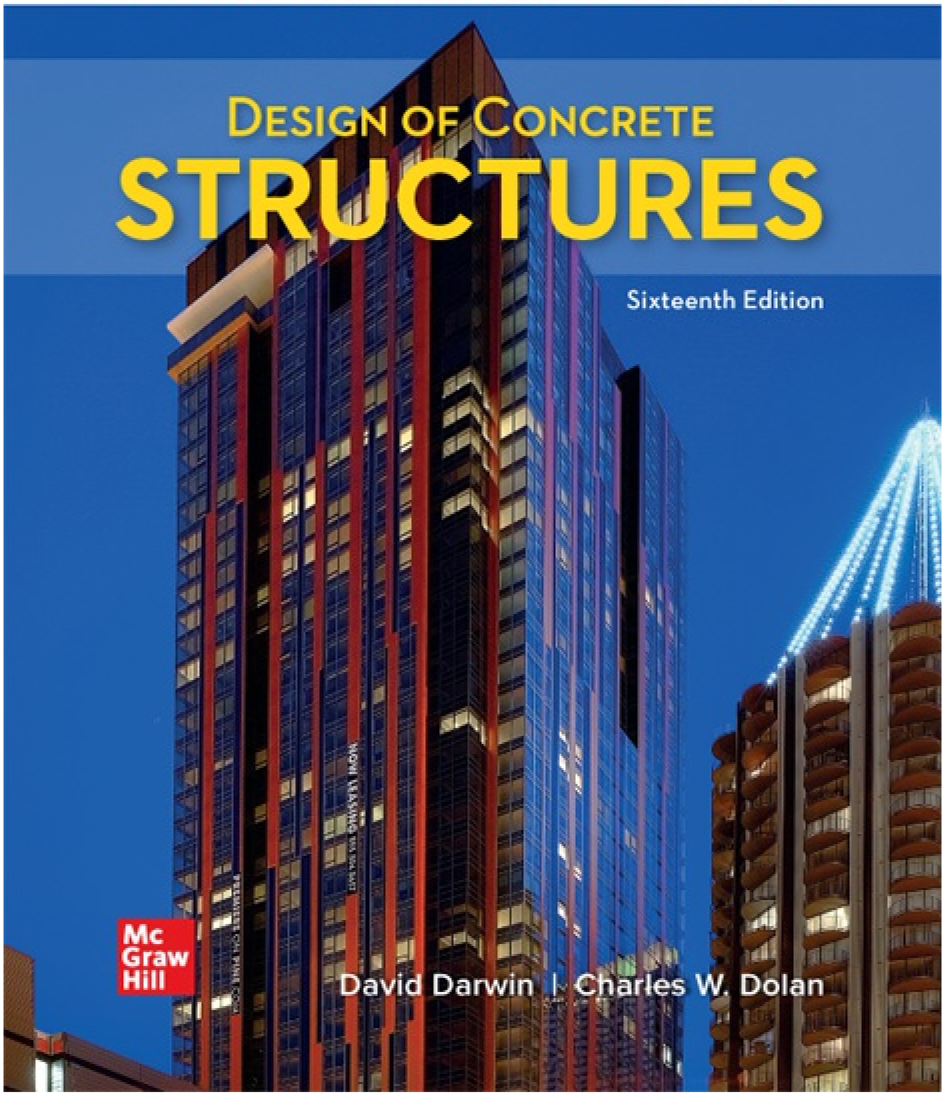

DESIGN of CONCRETE STRUCTURES

Sixteenth Edition

David Darwin

Ph.D., P.E., Distinguished Member of ASCE
Honorary Member of ACI, Fellow of SEI
Deane E. Ackers Distinguished Professor and Chair of Civil, Environmental & Architectural Engineering
University of Kansas

Charles W. Dolan

Ph.D., P.E., Honorary Member of ACI
Fellow of PCI

H. T. Person
Professor of Engineering, Emeritus
University of Wyoming

McGraw Hill

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Contents

Page viii

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About the Authors

David Darwin has been a member of the faculty at the University of Kansas since 1974, where he has served as director of the Structural Engineering and Materials Laboratory since 1982 and currently chairs the Department of Civil, Environmental, and Architectural Engineering. He was appointed the Deane E. Ackers Distinguished Professor of Civil Engineering in 1990. Dr. Darwin served as President of the American Concrete Institute (ACI) in 2007–2008 and is a member and past chair of ACI Committees 224 on Cracking and 408 on Bond and Development of Reinforcement. He is also a member of ACI Committee 318 Building Code for Concrete Structures and ACI-ASCE Committee 445 on Shear and Torsion. Dr. Darwin is an acknowledged expert on concrete crack control and bond between steel reinforcement and concrete. He received the ACI Arthur R. Anderson Award for his research efforts in plain and reinforced concrete, the ACI Structural Research Award, the ACI Joe W. Kelly Award for his contributions to teaching and design, the ACI Foundation—Concrete Research Council Arthur J. Boase Award for his research on reinforcing steel and concrete cracking, and the Concrete Research Council Robert E. Philleo Award for concrete material research and bridge construction practices. He has also received a number of awards from the American Society of Civil Engineers, including the Walter L. Huber Civil Engineering Research Prize, the Moisseiff Award, and the State-of-the-Art of Civil Engineering Award twice, the Richard R. Torrens Award, and the Dennis L. Tewksbury Award, and has been honored for his teaching by both undergraduate and graduate students at the University of Kansas. He is past editor of the ASCE *Journal of Structural Engineering.* Professor Darwin is a Distinguished Member of ASCE, an Honorary Member of ACI, and a Fellow of the Structural Engineering Institute of ASCE. He is a licensed professional engineer and serves as a consultant in the fields of concrete materials and structures. He has been honored with the Distinguished Alumnus Award from the University of

Illinois Civil and Environmental Engineering Alumni Association. Between his M.S. and Ph.D. degrees, he served four years with the U.S. Army Corps of Engineers. He received the B.S. and M.S. degrees from Cornell University in 1967 and 1968 and the Ph.D. from the University of Illinois at Urbana-Champaign in 1974.

Charles W. Dolan is a consulting engineer and emeritus faculty member of the University of Wyoming. At the University of Wyoming from 1991 to 2012, he served as Department Head from 1998 to 2001 and as the first H. T. Person Chair of Engineering from 2002 to 2012, for which he received the University of Wyoming's John P. Ellbogen lifetime teaching award. A member of American Concrete Institute (ACI) Committee 318 Building Code for Concrete Structures for 17 years, he has chaired the Building Code Subcommittees on Prestressed Concrete and Code Reorganization. He has served as chair of the ACI Technical Activities Committee, ACI Committee 358 on Transit Guideways, and ACI-ASCE Committee 423 on Prestressed Concrete. A practicing engineer for over 40 years, including 20 years at Berger/ABAM, he was the project engineer on the Walt Disney World Monorail, the Detroit Downtown Peoplemover guideway, and the original Dallas–Fort Worth Airport transit system guideway. He developed the conceptual design of the Vancouver BC SkyTrain structure and the Dubai Palm Island monorail. He received the ASCE T. Y. Lin Award for outstanding contributions to the field of prestressed concrete, the ACI Arthur R. Anderson award for advancements in the design of reinforced and prestressed concrete structures, and the Prestress/Precast Concrete Institute's (PCI) Martin P. Korn award for advances in design and research in prestressed concrete. An Honorary Member of ACI and a Fellow of PCI, he is internationally recognized as a leader in the design of specialty transit structures and development of fiber-reinforced polymers for concrete reinforcement. Dr. Dolan is a registered professional engineer and lectures widely on the design and behavior of structural concrete. He received his B.S. from the University of Massachusetts in 1965 and his M.S. and Ph.D. from Cornell University in 1967 and 1989.

Preface

The sixteenth edition of *Design of Concrete Structures* continues the dual objectives of establishing a firm understanding of the behavior of structural concrete and of developing proficiency in the methods of design practice. It is generally recognized that mere training in special design skills and codified procedures is inadequate for a successful career in professional practice. As new research becomes available and new design methods are introduced, these procedures are subject to frequent changes. To understand and keep abreast of these rapid developments and to engage safely in innovative design, the engineer needs a thorough grounding in the fundamental performance of concrete and steel as structural materials and in the behavior of reinforced concrete members and structures. At the same time, the main business of the structural engineer is to design structures safely, economically, and efficiently. Consequently, with this basic understanding as a firm foundation, familiarity with current design procedures is essential. This edition, like the preceding ones, addresses both needs.

The text presents the basic mechanics of structural concrete and methods for the design of individual members subjected to bending, shear, torsion, and axial forces. It additionally addresses in detail applications of the various types of structural members and systems, including an extensive presentation of slabs, beams, columns, walls, footings, retaining walls, and the integration of building systems.

The 2019 ACI Building Code, which governs design practice in most of the United States and serves as a model code in many other countries, underwent a number of significant changes, many due to increases in the specified strengths of reinforcing steels that can used for building construction.

Changes of note include the addition of Grade 100 steel for use as principal reinforcement for gravity and lateral loads and the recognition that changes were needed in the Code, even for Grade 80 reinforcement. The use of steels with grades above 60, long the standard in U.S. practice, has led to changes in the approaches to both strength and serviceability, including the limits on both maximum and minimum reinforcement; development lengths of straight, hooked, and headed reinforcement; and requirements for the effective moment of inertia when calculating deflections. Shear design has changed through the addition of a size effect term that recognizes that shear stress at failure decreases as member depth increases. Inclusion of the size effect affects foundation walls, as well as beams and slabs—a point that is highlighted in this edition. The techniques used for two-way slab design were deleted from the 2019 ACI Building Code with the understanding that those techniques would be covered by textbooks. That information has been retained in Chapters 13, 22, and 23. Finally, the requirements for the strut-and-tie method have been updated.

In addition to changes in the ACI Code, the text also includes the modified compression field theory method of shear design presented in the 2017 edition of the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications*. Chapters on yield line and strip methods, on the McGraw-Hill Education website in the previous edition, have been returned to the printed version of the text.

A strength of the text is the analysis chapter, which includes load combinations for use in design, a description of envelope curves for moment and shear, guidelines for proportioning members under both gravity and lateral loads, and procedures for developing preliminary designs of reinforced concrete structures. The chapter also includes the ACI moment and shear coefficients.

Present-day design is performed using computer programs, either general-purpose commercially available software or individual programs written for special needs. Procedures given throughout the book guide the student and engineer through the increasingly complex methodology of design, with the emphasis on understanding the design process. Once mastered, these procedures are easily converted into flow charts to aid in preparing design aids or to validate commercial computer program output.

The text is suitable for either a one- or two-semester course in the design of concrete structures. If the curriculum permits only a single course, probably taught in the fourth undergraduate year, the following will provide a good basis: the introduction and treatment of materials found in Chapters 1 through 3; the material on flexure, shear, and anchorage in Chapters 4, 5, and 6; Chapter 7 on serviceability; Chapter 9 on short columns; the introduction to one-way slabs found in Chapter 12; and footings, Chapter 15. Time may or may not permit classroom coverage of frame analysis or building systems, Chapters 11 and 19, but these could well be assigned as independent reading, concurrent with the earlier work of the course. In the authors' experience, such complementary outside reading tends to enhance student motivation.

The text is more than adequate for a second course, most likely taught in the senior year or first year of graduate study. The authors have found that this is an excellent opportunity to provide students with a more general understanding of reinforced concrete structural design, often beginning with analysis and building systems, Chapters 11 and 19, followed by the increasingly important behavioral topics of torsion, Chapter 8; slender columns, Chapter 10; the strut-and-tie method of Chapter 17; and the design and detailing of joints, Chapter 18. It should also offer an opportunity for a much-expanded study of slabs, including Chapter 13, plus the methods for slab analysis and design based on plasticity theory found in Chapters 23 and 24, yield line analysis, and the strip method of design. Other topics appropriate to a second course include retaining walls, Chapter 16, and the introduction to earthquake-resistant design in the expanded Chapter 20. Prestressed concrete in Chapter 22 is sufficiently important to justify a separate course in conjunction with anchoring to concrete, Chapter 21, and strut-and-tie methods, Chapter 17. If time constraints do not permit this, Chapter 22 provides an introduction and can be used as the text for a one-credit-hour course.

At the end of each chapter, the user will find extensive reference lists, which provide an entry into the literature for those wishing to increase their knowledge through individual study. For professors, the Instructor's Solution Manual is available online at the McGraw-Hill Education website.

A word must be said about units. In the United States, customary inch-pound units remain prominent. Accordingly, inch-pound units are used throughout the text, although some graphs and basic data in Chapter 2 are given in dual units. Appendix B gives the SI equivalents of inch-pound units.

A brief historical note may be of interest. This book is the sixteenth edition of a textbook originated in 1923 by Leonard C.

Urquhart and Charles E. O’Rourke, both professors of structural engineering at Cornell University. Over its remarkable 97-year history, new editions have kept pace with research, improved materials, and new methods of analysis and design. The second, third, and fourth editions firmly established the work as a leading text for elementary courses in the subject area. Professor George Winter, also of Cornell, collaborated with Urquhart in preparing the fifth and sixth editions. Winter and Professor Arthur Nilson were responsible for the seventh, eighth, and ninth editions, which substantially expanded both the scope and the depth of the presentation. The tenth, eleventh, and twelfth editions were prepared by Professor Nilson subsequent to Professor Winter’s passing in 1982.

Professor Nilson was joined by Professor David Darwin of the University of Kansas and by Professor Charles Dolan of the University of Wyoming for the thirteenth, fourteenth, and fifteenth editions, although Professor Nilson passed away prior to completion of the fifteenth. Like Professors Winter and Nilson, the current authors have been deeply involved in research and teaching in the fields of reinforced and prestressed concrete, as well as professional Code-writing committees, and have spent significant time in professional practice, invaluable in developing the perspective and structural judgment that sets this book apart.

Special thanks are due to the McGraw-Hill Education project team, notably, Sarah Paratore, Sue Nodine, Carey Lange, and Jane Mohr.

We gladly acknowledge our indebtedness to the original authors. Although it is safe to say that neither Urquhart or O’Rourke would recognize much of the detail and that Winter would be impressed by the many changes, the approach to the subject and the educational philosophy that did so much to account for the success of the early editions would be familiar. The imprint of Arthur Nilson—our longstanding mentor, colleague, and friend—remains clear in the organization and approach taken to the material in this text.

David Darwin
Charles W. Dolan

1 Introduction

1.1 CONCRETE, REINFORCED CONCRETE, AND PRESTRESSED CONCRETE

Concrete is a stonelike material obtained by permitting a carefully proportioned mixture of cement, sand and gravel or other coarse aggregate, and water to harden in forms of the shape and dimensions of the desired structure. The bulk of the material consists of fine and coarse aggregate. Cement and water interact chemically to bind the aggregate particles into a solid mass. Additional water, over and above that needed for this chemical reaction, is necessary to give the mixture the workability that enables it to fill the forms and surround the embedded reinforcing steel prior to hardening. Concretes with a wide range of properties can be obtained by appropriate adjustment of the proportions of the constituent materials. Special cements (such as high early strength cements), special aggregates (such as various lightweight or heavyweight aggregates), admixtures (such as plasticizers, air-entraining agents, silica fume, and fly ash), and special curing methods (such as steam-curing) permit an even wider variety of properties to be obtained.

These properties depend to a very substantial degree on the proportions of the mixture, on the thoroughness with which the various constituents are intermixed, and on the conditions of humidity and temperature in which the mixture is maintained from the moment it is placed in the forms until it is fully hardened. The process of controlling conditions after placement is known as *curing*. To protect against the unintentional production of substandard concrete, a high degree of skillful control and supervision is necessary throughout the process, from the proportioning by weight of the individual components, through mixing and placing, until the completion of curing.

The factors that make concrete a universal building material are so pronounced that it has been used, in more primitive kinds and ways than at present, for thousands of years, starting with lime mortars from 12,000 to 6000 BCE in Crete, Cyprus, Greece, and the Middle East. The facility with which, while plastic, it can be deposited and made to fill forms or molds of almost any practical shape is one of these factors. Its high fire and weather resistance is an evident advantage. Most of the constituent materials, with the exception of cement and additives, are usually available at low cost locally or at small distances from the construction site. Its compressive strength, like that of natural stones, is high, which makes it suitable for members primarily subject to compression, such as columns and arches. On the other hand, again as in natural stones, it is a relatively brittle material whose tensile strength is low compared with its compressive strength. This prevents its economical use as the sole building material in structural members that are subject to tension either entirely (such as in tie-rods) or over part of their cross sections (such as in beams or other flexural members).

To offset this limitation, it was found possible, in the second half of the nineteenth century, to use steel with its high tensile strength to reinforce concrete, chiefly in those places where its low tensile strength would limit the carrying capacity of the member. The reinforcement, usually round steel rods with appropriate surface deformations to provide interlocking, is placed in the forms in advance of the concrete. When - completely surrounded by the hardened concrete mass, it forms an integral part of the member. The resulting combination of two materials, known as *reinforced concrete*, combines many of the advantages of each: the relatively low cost, good weather and fire resistance, good compressive strength, and excellent formability of concrete and the high tensile strength and much greater ductility and toughness of steel. It is this combination that allows the almost unlimited range of uses and possibilities of reinforced concrete in the construction of buildings, bridges, dams, tanks, reservoirs, and a host of other structures.

It is possible to produce steels, at relatively low cost, whose yield strength is 3 to 4 times and more that of ordinary reinforcing steels. Likewise, it is possible to produce concrete 4 to 5 times as strong in compression as the more ordinary concretes. These high-strength materials offer many advantages, including smaller member cross sections, reduced dead load, and longer spans. However, there are limits to the strengths of the constituent materials beyond which certain problems arise. To be sure, the strength of such a member would increase roughly in proportion to those of the materials. However, the high strains that result from the high stresses that would otherwise be permissible would lead to large deformations and consequently large deflections of such members under ordinary loading conditions. Equally important, the large strains in such high-strength reinforcing steel would induce large cracks in the surrounding low tensile strength concrete, cracks that not only would be unsightly but also could significantly reduce the durability of the structure. This limits the useful yield strength of high-strength reinforcing steel to 100 ksi[‡] according to many codes and specifications; 60 and 80 ksi steel is most commonly used.

Construction known as *prestressed concrete*, however, does use steels and concretes of very high strength in combination. The steel, in the form of wires, strands, or bars, is embedded in the concrete under high tension that is held in equilibrium by compressive stresses in the concrete after hardening. Because of this precompression, the concrete in a flexural member will crack on the tension side at a much larger load than when not so precompressed. Prestressing greatly reduces both the deflections and the tensile cracks at ordinary loads in such structures and thereby enables these high-strength materials to be used effectively. Prestressed concrete has extended, to a very significant extent, the range of spans of structural concrete and the types of structures for which it is suited.

1.2 STRUCTURAL FORMS

The figures that follow show some of the principal structural forms of reinforced concrete. Pertinent design methods for many of them are discussed later in this volume.

Floor support systems for buildings include the monolithic slab-and-beam floor shown in Fig. 1.1, the one-way joist system of Fig. 1.2, and the flat plate floor, without beams or girders, shown in Fig. 1.3. The flat slab floor of Fig. 1.4, frequently used for more heavily loaded buildings, is similar to the flat plate floor, but makes use of increased slab thickness in the vicinity of the columns, as well as flared column tops, to reduce stresses and increase strength in the support region. The choice among these and other systems for floors and roofs depends upon functional requirements, loads, spans, and permissible member depths, as well as on cost and esthetic factors.

FIGURE 1.1

One-way reinforced concrete floor slab with monolithic supporting beams. (*Courtesy of Portland Cement Association*)

FIGURE 1.2

One-way joist floor system, with closely spaced ribs supported by monolithic concrete beams; transverse ribs provide for lateral distribution of localized loads. (*Courtesy of Portland Cement Association*)

FIGURE 1.3

Flat plate floor slab, carried directly by columns without beams or girders. (*Courtesy of Portland Cement Association*)

FIGURE 1.4

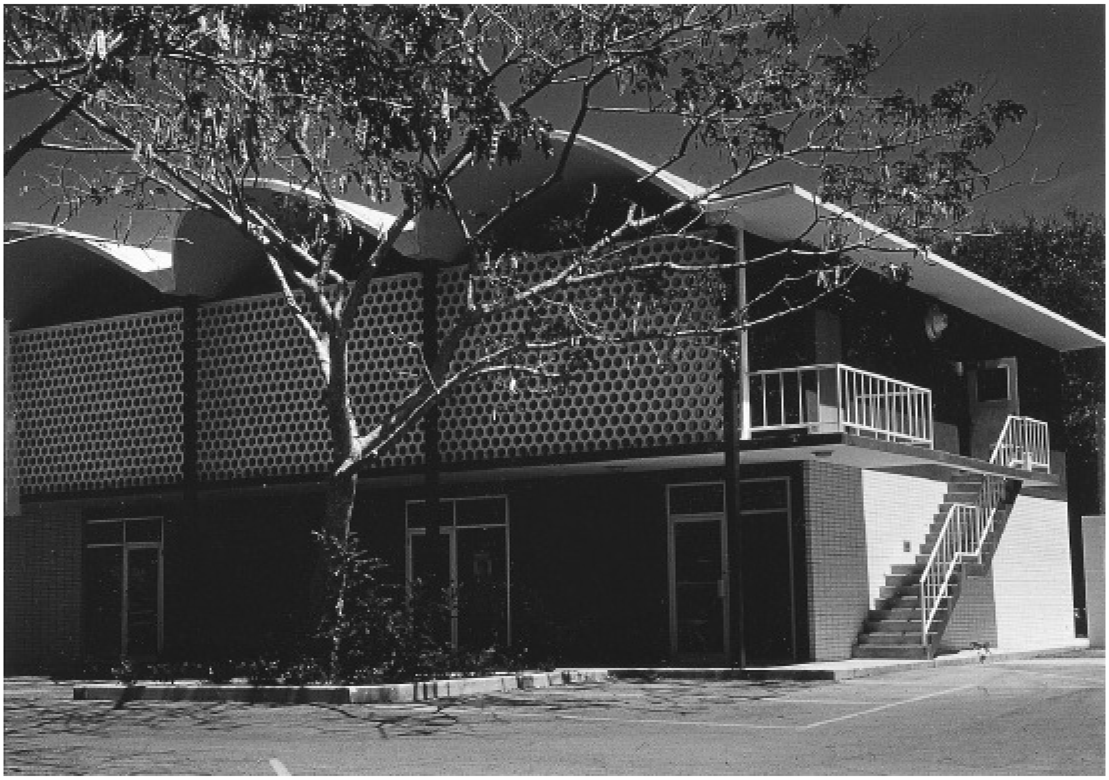
Flat slab floor, without beams but with slab thickness increased at the columns and with flared column tops to provide for local concentration of forces. (*Courtesy of Portland Cement Association*)

Where long clear spans are required for roofs, concrete shells permit use of extremely thin surfaces, often thinner, relatively, than an eggshell. The folded plate roof of Fig. 1.5 is simple to form because it is composed of flat surfaces; such roofs have been employed for spans of 200 ft and more. The cylindrical shell of Fig. 1.6 is also relatively easy to form because it has only a single curvature; it is similar to the folded plate in its structural behavior and range of spans and loads. Shells of this type were once quite popular in the United States and remain popular in other parts of the world.

FIGURE 1.5

Folded plate roof of 125 ft span that, in addition to carrying ordinary roof loads, carries the second floor as well using a system of cable hangers; the ground floor is kept free of columns. (*Photograph by Arthur H. Nilson*)

FIGURE 1.6
Cylindrical shell roof providing column-free interior space. (*Photograph by Arthur H. Nilson*)



Doubly curved shell surfaces may be generated by simple mathematical curves such as circular arcs, parabolas, and hyperbolas, or they may be composed of complex combinations of shapes. Hemispherical concrete domes are commonly used for storage of bulk materials. The dome shown in Fig. 1.7 is for storage of dry cement, and the piping around the perimeter is for the pneumatic movement of the cement. Domed structures are commonly constructed using shotcrete, a form of concrete that is sprayed onto a liner and requires formwork or backing on only one side. The dome in Fig. 1.7 was constructed by inflating a membrane, spraying insulation on the membrane, placing the reinforcement on the insulation, then spraying the concrete on both sides of the insulation to the prescribed thickness using the insulation as a backing form, as shown in Fig. 1.8.

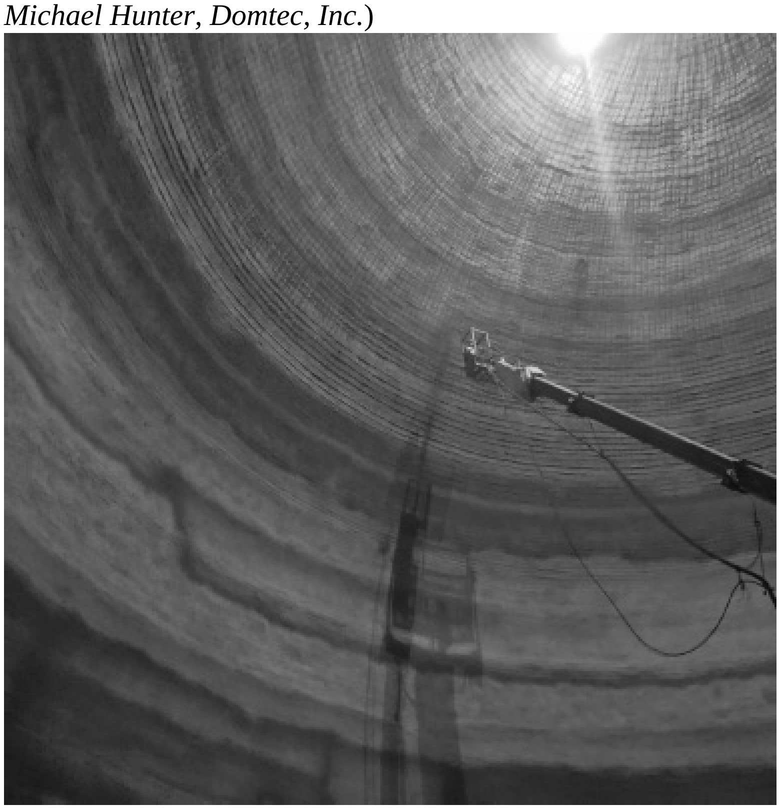
Piers and wharf facilities (shown in Fig. 1.7), silos, waters tanks, reservoirs, and other industrial facilities are commonly constructed of reinforced or prestressed concrete.

FIGURE 1.7

Hemispherical cement storage dome in New Zealand. (*Photograph courtesy of Michael Hunter, Domtec, Inc.*)

FIGURE 1.8

Shotcrete being applied to the interior of a dome structure. (*Photograph courtesy of Michael Hunter, Domtec, Inc.*)



Bridge design has provided the opportunity for some of the most challenging and creative applications of structural engineering. The award-winning Napoleon Bonaparte Broward Bridge, shown in Fig. 1.9, is a six-lane, cable-stayed structure that spans St. John's River at Dame Point, Jacksonville, Florida. It has a 1300 ft center span. Figure 1.10 shows the Bennett Bay Centennial Bridge, a four-span continuous, segmentally cast-in-place box girder structure. Special attention was given to esthetics in this award-winning design. The spectacular Natchez Trace Parkway Bridge in Fig. 1.11, a two-span arch structure using hollow precast concrete elements, carries a two-lane highway 155 ft above the valley floor.

FIGURE 1.9
Napoleon Bonaparte Broward Bridge, with a 1300 ft center span at Dame Point, Jacksonville, Florida. (*HNTB Corporation, Kansas City, Missouri*)

FIGURE 1.10
Bennett Bay Centennial Bridge, Coeur d'Alene, Idaho, a four-span continuous concrete box girder structure of length 1730 ft. (*HNTB Corporation, Kansas City, Missouri*)

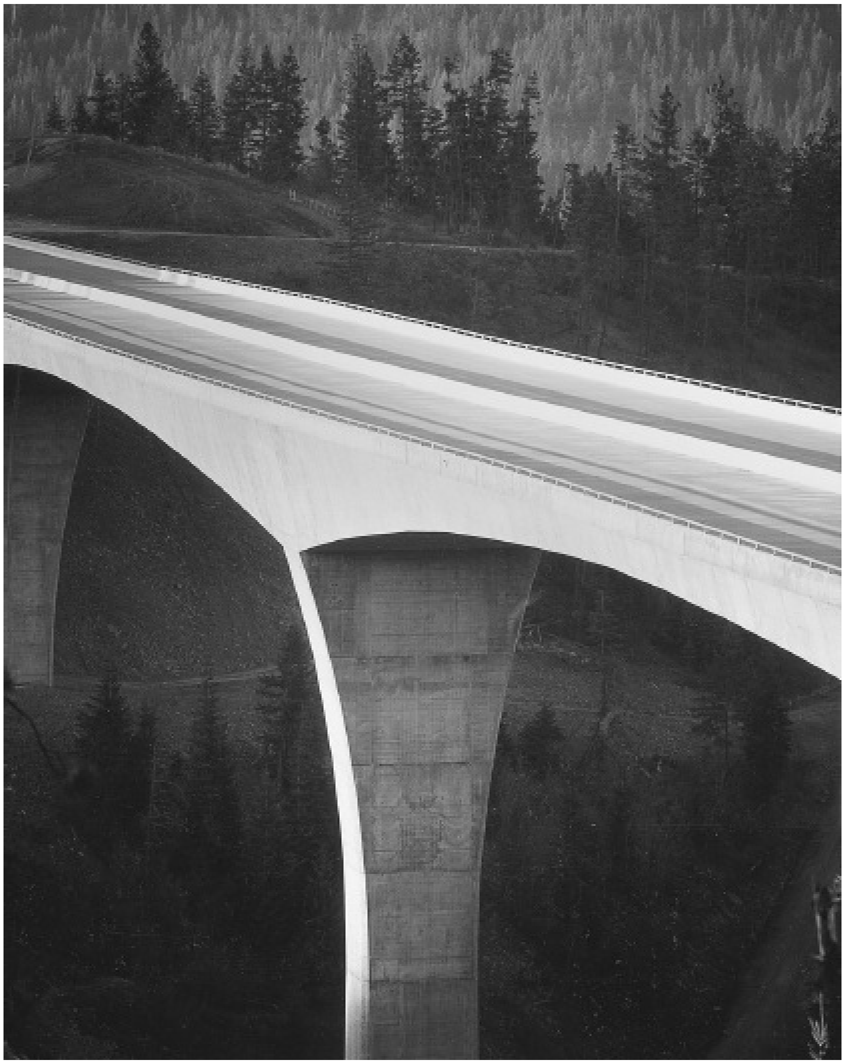


FIGURE 1.11

Natchez Trace Parkway Bridge near Franklin, Tennessee, an award-winning two-span concrete arch structure rising 155 ft above the valley floor. (*Designed by Figg Bridge Group*)

Buildings clad in glass or other fascia materials do not immediately indicate the underlying structural framing. The *Premiere on Pine* building in downtown Seattle is a case in point. The 42-story building contains condominiums, underground parking, and a hotel-type sky lounge. The 450,000 square foot flat slab cast-in-place concrete construction uses 15,000 psi concrete for columns to increase available floor space and to resist gravity and earthquake loads; see Fig. 1.12.

FIGURE 1.12

Premier on Pine under construction. The cover photo is the competed building. (*Photograph provided by Cary Kopczynski and Company, Structural Engineers*)

The structural forms shown in Figs. 1.1 to 1.12 hardly constitute a complete inventory but are illustrative of the shapes appropriate to the properties of reinforced or prestressed concrete. They illustrate the adaptability of the material to a great variety of one-dimensional (beams, girders, columns), two-dimensional (slabs, arches, rigid frames), and three-dimensional (shells, tanks) structures and structural components. This variability allows the shape of the structure to be adapted to its function in an economical manner, and furnishes the architect and design engineer with a wide variety of possibilities for esthetically satisfying structural solutions.

1.3 LOADS

Loads that act on structures can be divided into three broad categories: dead loads, live loads, and environmental loads.

Dead loads are those that are constant in magnitude and fixed in location throughout the lifetime of the structure. Usually the major part of the dead load is the weight of the structure itself. This can be calculated with good accuracy from the design configuration, dimensions of the structure, and density of the material. For buildings, floor fill, finish floors, and plastered ceilings are usually included as dead loads, and an allowance is made for suspended loads such as piping and lighting fixtures. For bridges, dead loads may include wearing surfaces, sidewalks, and curbs, and an allowance is made for piping and other suspended loads.

Live loads consist chiefly of occupancy loads in buildings and traffic loads on bridges. They may be either fully or partially in place or not present at all, and may also change in location. Their magnitude and distribution at any given time are uncertain, and even their maximum intensities throughout the lifetime of the structure are not known with precision. The minimum live loads for which the floors and roof of a building should be designed are usually specified in the building code that governs at the site of construction. Representative values of minimum live loads to be used in a wide variety of buildings are found in *Minimum Design Loads and Other Associated Criteria for Buildings and Other Structures* (Ref. 1.1), a portion of which is reprinted in Table 1.1. The table gives uniformly distributed live loads for various types of occupancies; these include impact and concentrated load provisions where necessary. These loads are expected maxima and considerably exceed average values.

In addition to these uniformly distributed loads, it is recommended that, as an alternative to the uniform load, floors be designed to support safely certain concentrated loads if these produce a greater stress. For example, according to Ref. 1.1, office floors are to be designed to carry a load of 2000 lb distributed over an area 2.5 ft square (6.25 ft^2), to allow for heavy equipment, and stair treads must safely support a 300 lb load applied on the center of the tread. Certain reductions are often permitted in live loads for members supporting large areas with the understanding that it is unlikely that the entire area would be fully loaded at one time (Refs. 1.1 and 1.2).

Tabulated live loads cannot always be used. The type of occupancy should be considered and the probable loads computed as accurately as possible. Warehouses for heavy storage may be designed for loads as high as 500 psf or more; unusually heavy operations in manufacturing buildings may require an increase in the 250 psf value specified in Table 1.1; special provisions must be made for all definitely located heavy concentrated loads.

TABLE 1.1
Minimum uniformly distributed live loads in pounds per square foot (psf)

Occupancy or Use	Live Load, psf
Apartments (see Residential)	
Access floor systems	
Office use	50
Computer use	100
Armories and drill rooms[a]	150
Assembly areas and theaters	
Fixed seats (fastened to floor)[a]	60
Lobbies[a]	100
Movable seats[a]	100
Platforms (assembly)[a]	100
Stage floors[a]	150
Balconies and decks[b]	
Catwalks for maintenance access	40
Corridors	
First floor	100
Other floors, same as occupancy served except as indicated	
Dining rooms and restaurants[a]	100
Dwellings (see Residential)	
Fire escapes	100
On single-family dwellings only	40
Garages (passenger vehicles only)[a,c,d]	40
Trucks and buses[c]	
Dances halls	100
Gymnasiums[a]	100
Residential	
One- and two-family dwellings	
Uninhabitable attics without storage[f]	10
Uninhabitable attics with storage[g]	20
Habitable attics and sleeping areas	30
All other areas except stairs	40
All other residential occupancies, hotels, and multifamily houses	
Private rooms and corridors serving them	40
Public rooms and corridors serving them	100
Roofs	
Ordinary flat, pitched, and curved roofs[h]	20
Roofs used for roof gardens	100
Roofs used for assembly purposes	
Roofs used for other occupancies[i]	
Awnings and canopies	
Fabric construction supported by a lightweight rigid skeleton structure	5
All other construction	20
Hospitals	
Operating rooms, laboratories	60
Patient rooms	40
Corridors above first floor	80
Hotels (see Residential)	
Libraries	
Reading rooms	60
Stack rooms[a,e]	150
Corridors above first floor	80
Manufacturing	
Light[a]	125
Heavy[a]	250
Office buildings	
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	
Lobbies and first floor corridors	100
Offices	50
Corridors above first floor	80
Penal institutions	
Cell blocks	40
Corridors	100
Recreational uses	
Bowling alleys[a]	75
Schools	
Classrooms	40
Corridors above first floor	80
First floor corridors	100
Sidewalks, vehicular driveways, and yards, subject to trucking[a,j]	250
Stairs and exit-ways	100
One- and two-family residences only	40
Storage areas above ceilings	20
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)	
Light[a]	125
Heavy[a]	250
Stores	
Retail	
First floor	100
Upper floors	73
Wholesale, all floors[a]	125
Walkways and elevated platforms (other than exit-ways)	60
Yards and terraces, pedestrians	100

[a] Live load reduction for this use is not permitted unless specific exceptions apply.

[b] 1.5 times live load for area served. Not required to exceed 100 psf.

[c] Floors of garages or portions of a building used for storage of motor vehicles shall be designed for the uniformly distributed live loads of this table or for concentrated loads specified in Ref. 1.1.

[d] Design for trucks and buses shall be in accordance with Ref. 1.3; however, provisions for fatigue and dynamic load are not required.

[e] The loading applies to stack room floors that support nonmobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 90 in.; (2) the nominal shelf depth shall not exceed 12 in. for each face; and (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 36 in. wide.

[f] See Ref. 1.1 for description of uninhabitable attic areas without storage. This live load need not be assumed to act concurrently with any other live load requirement.

[g] See Ref. 1.1 for description of uninhabitable attic areas with storage and where this provision applies.

[h] Where uniform roof live loads are reduced to less than 20 psf in accordance with Section 4.8.2 of Ref. 1.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable load effect.

[i] Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

[j] Other uniform loads in accordance with an approved method that contains provisions for truck loadings shall also be considered where appropriate.

Data Source: *Minimum Design Loads and Other Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7-16). American Society of Civil Engineers, Reston, VA, 2010.

Live loads for highway bridges are specified by the American Association of State Highway and Transportation Officials (AASHTO) in its *LRFD Bridge Design Specifications* (Ref. 1.3). For railway bridges, the American Railway Engineering and Maintenance-of-Way Association (AREMA) has published the *Manual of Railway Engineering* (Ref. 1.4), which specifies traffic loads.

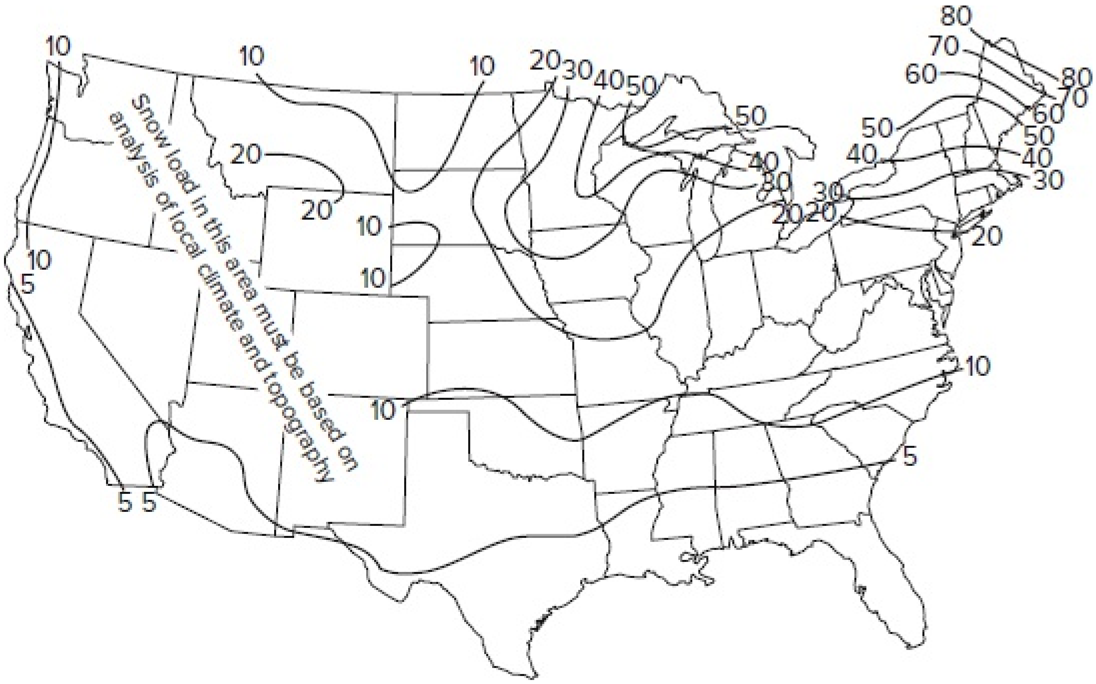
Environmental loads consist mainly of snow loads, wind pressure and suction, earthquake load effects (that is, inertia forces caused by earthquake motions), soil and hydraulic pressures on subsurface portions of structures, loads from possible ponding of rainwater on flat surfaces, and forces caused by temperature differentials. Like live loads, environmental loads at any given time are uncertain in both magnitude and distribution. Reference 1.1 contains much information on environmental loads, which is often modified locally depending, for instance, on local climatic or seismic conditions.

Figure 1.13, from the 1972 edition of Ref. 1.1, gives snow loads

for the continental United States and is included here for illustration only. The 2016 edition of Ref. 1.1 gives much more detailed information. In either case, specified values represent not average values, but expected upper limits. A minimum roof load of 20 psf is often specified to provide for construction and repair loads and to ensure reasonable stiffness.

FIGURE 1.13

Snow load in pounds per square foot (psf) on the ground, 50-year mean recurrence interval. (*Minimum Design Loads for Buildings and Other Structures, ANSI A58.1–1972, American National Standards Institute, New York, 1972.*)



Much progress has been made in developing rational methods for predicting horizontal forces on structures due to wind and seismic action. Reference 1.1 summarizes current thinking regarding wind forces and earthquake loads. Reference 1.5 presents detailed recommendations for lateral forces from earthquakes.

Reference 1.1 specifies design wind pressures per square foot of vertical wall surface. Depending upon locality, these equivalent static forces vary from about 10 to 50 psf. Factors include basic wind speed, exposure (urban vs. open terrain, for example), height of the structure, the importance of the structure (that is, consequences of failure), and gust effect factors to account for the fluctuating nature of the wind and its interaction with the structure.

Seismic forces may be found for a particular structure by elastic or inelastic dynamic analysis, considering expected ground accelerations and the mass, stiffness, and damping characteristics of the construction. In less seismically active areas, the design is often based on equivalent static forces calculated from provisions such as those of Refs. 1.1 and 1.5. The base shear is found by considering such factors as location, type of structure and its occupancy, total dead load, and the particular soil condition. The total lateral force is distributed to floors over the entire height of the structure in such a way as to approximate the distribution of forces obtained from a dynamic analysis.

1.4 SERVICEABILITY, STRENGTH, AND STRUCTURAL SAFETY

To serve its purpose, a structure must be safe against collapse and serviceable in use. Serviceability requires that deflections be adequately small; that cracks, if any, be kept to tolerable limits; and that vibrations be minimized. Safety requires that the strength of the structure be adequate for all loads that may foreseeably act on it. If the strength of a structure, built as designed, could be predicted accurately, and if the loads and their internal effects (bending moments, shears, axial forces, and torsional moments) were known accurately, safety could be ensured by providing a carrying capacity just barely in excess of the known loads. However, there are a number of sources of uncertainty in the analysis, design, and construction of reinforced concrete structures. These sources of uncertainty, which require a definite margin of safety, may be listed as follows:

1. Actual loads may differ from those assumed.
2. Actual loads may be distributed in a manner different from that assumed.
3. The assumptions and simplifications inherent in any analysis may result in calculated load effects—moments, shears, etc.—different from those that, in fact, act in the structure.
4. The actual structural behavior may differ from that assumed, owing to imperfect knowledge.
5. Actual member dimensions may differ from those specified.
6. Reinforcement may not be in its proper position.
7. Actual material strength may be different from that specified.

In the establishment of safety requirements, consideration must be given to the consequences of failure. In some cases, a failure would be merely an inconvenience. In other cases, loss of life and significant loss of property may be involved. A further consideration should be the nature of the failure, should it occur. A gradual failure with ample warning permitting remedial measures is preferable to a sudden, unexpected collapse.

It is evident that the selection of an appropriate margin of safety is not a simple matter. However, progress has been made toward rational safety provisions in design codes (Refs. 1.6 to 1.11).

a. Variability of Loads

Since the maximum load that occurs during the life of a structure is uncertain, it can be considered a random variable. In spite of this uncertainty, the engineer must provide an adequate structure. A probability model for the maximum load can be devised by means of a probability density function for loads (Ref. 1.8), as represented by the frequency curve of Fig. 1.14*a*. The exact form of this distribution curve, for a particular type of loading such as office loads, can be determined only on the basis of statistical data obtained from large-scale load surveys. A number of such surveys have been completed. For types of loads for which such data are scarce, fairly reliable information can be obtained from experience, observation, and judgment.

For such a frequency curve (Fig. 1.14*a*), the area under the curve between two abscissas, such as loads Q_1 and Q_2, represents the probability of occurrence of loads Q of magnitude $Q_1 < Q < Q_2$. A specified service load Q_d for design is selected conservatively in the upper region of Q in the distribution curve, as shown. The probability of occurrence of loads larger than Q_d is then given by the shaded area to the right of Q_d. It is seen that this specified service load is considerably larger than the mean load Q acting on the structure. This mean load is much more typical of average load conditions than the design load Q_d.

b. Strength

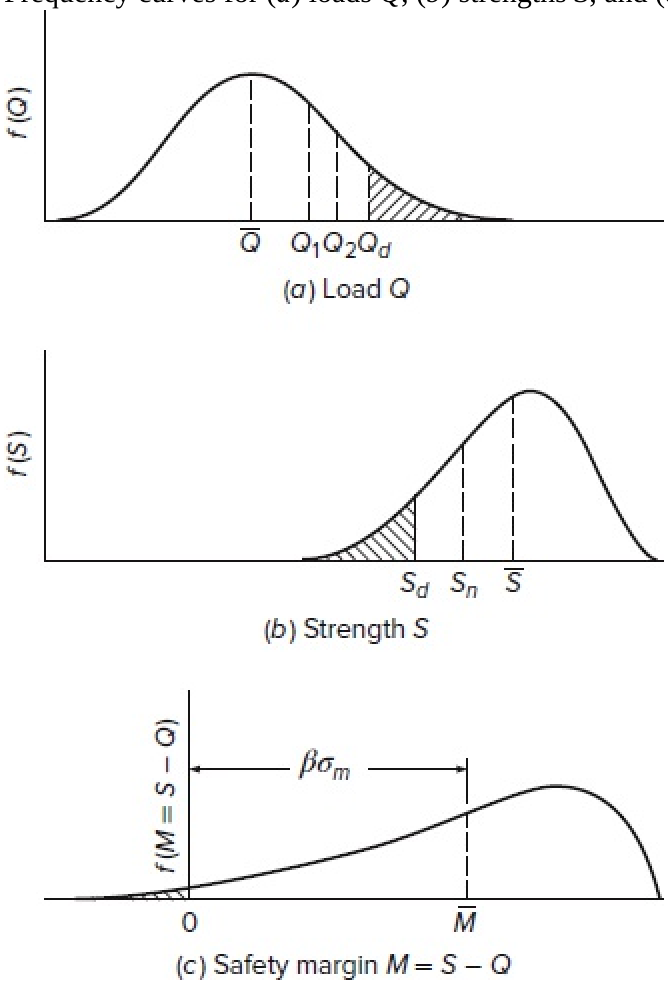
The strength of a structure depends on the strength of the materials from which it is made. For this purpose, minimum material strengths are specified in standardized ways. Actual material strengths cannot be known precisely and therefore also constitute random variables (see Section 2.6). Structural strength depends, furthermore, on the care with which a structure is built, which in turn reflects the quality of supervision and inspection. Member sizes may differ from specified dimensions, reinforcement may be out of position, poorly placed concrete may show voids, etc.

The strength of the entire structure or of a population of repetitive -

structures, such as highway overpasses, can also be considered a random variable with a probability density function of the type shown in Fig. 1.14*b*. As in the case of loads, the exact form of this function cannot be known but can be approximated from known data, such as statistics of actual, measured materials and member strengths and similar information. Considerable information of this type has been, or is being, developed and used.

FIGURE 1.14

Frequency curves for (*a*) loads *Q*, (*b*) strengths *S*, and (*c*) safety margin *M*.



c. Structural Safety

A given structure has a *safety margin M* if

$$M = S - Q > 0 \tag{1.1}$$

that is, if the strength of the structure is larger than the load acting on it. Since S and Q are random variables, the safety margin $M = S - Q$ is also a random variable. A plot of the probability function of M may appear as in Fig. 1.14*c*. Failure occurs when M is less than zero. Thus, the probability of failure is represented by the shaded area in the figure.

Even though the precise form of the probability density functions for S and Q, and therefore for M, is not known, much can be achieved in the way of a rational approach to structural safety. One such approach is to require that the mean safety margin M be a specified number β of standard deviations σ_m above zero. It can be demonstrated that this results in the requirement that

$$\psi_s \bar{S} \geq \psi_L \bar{Q} \tag{1.2}$$

where ψ_s is a partial safety coefficient smaller than 1.0 applied to the mean strength and ψ_L is a partial safety coefficient larger than 1.0 applied to the mean load . The magnitude of each partial safety coefficient depends on the variance of the quantity to which it applies, S or Q, and on the chosen value of β, the reliability index of the structure. As a general guide, a value of the safety index β between 3 and 4 corresponds to a probability of failure of the order of 1:100,000 (Ref. 1.9). The value of β is often established by calibration against well-proved and established designs.

In practice, it is more convenient to introduce partial safety coefficients with respect to code-specified loads that considerably exceed average values, rather than with respect to mean loads as in Eq. (1.2); similarly, the partial safety coefficient for strength is applied to *nominal strength*[†] generally computed somewhat conservatively, rather than to mean strengths as in Eq. (1.2). A restatement of the safety requirement in these terms is

$$\phi S_n \geq \gamma Q_d \tag{1.3a}$$

in which ϕ is a *strength reduction factor* applied to nominal strength S_n and γ is a *load factor* applied to calculated or code-specified design loads Q_d. Furthermore, recognizing the differences in variability between, say, dead loads D and live loads L, it is both reasonable and easy to introduce different load factors for different types of loads. The preceding equation can thus be written

$$\phi\, S_n \geq \gamma_d\, D + \gamma_l\, L \tag{1.3b}$$

in which γ_d is a load factor somewhat greater than 1.0 applied to the calculated dead load D and γ_l is a larger load factor applied to the code-specified live load L. When additional loads, such as the wind load W, are to be considered, the reduced probability that maximum dead, live, and wind or other loads will act simultaneously can be incorporated by using modified load factors such that

$$\phi\, S_n \geq \gamma_{d_i}\, D + \gamma_{l_i}\, L + \gamma_{w_i}\, W + \ldots \tag{1.3c}$$

Present U.S. design codes follow the format of Eqs. (1.3*b*) and (1.3*c*).

1.5 DESIGN BASIS

The single most important characteristic of any structural member is its actual strength, which must be large enough to resist, with some margin to spare, all foreseeable loads that may act on it during the life of the structure, without failure or other distress. It is logical, therefore, to proportion members, that is, to select concrete dimensions and reinforcement, so that member strengths are adequate to resist forces resulting from certain hypothetical overload stages, significantly above loads expected actually to occur in service. This design concept is known as *strength design*.

For reinforced concrete structures at loads close to and at failure, Page 16 one or both of the materials, concrete and steel, are invariably in their nonlinear inelastic range. That is, concrete in a structural member reaches its maximum strength and subsequent fracture at stresses and strains far beyond the initial elastic range in which stresses and strains are fairly proportional. Similarly, steel close to and at failure of the member is usually stressed beyond its elastic domain into and even beyond the yield region. Consequently, the nominal strength of a member must be calculated on the basis of this inelastic behavior of the materials.

A member designed by the strength method must also perform in a satisfactory way under normal service loading. For example, beam deflections must be limited to acceptable values, and the number and width of flexural cracks at service loads must be controlled. Serviceability limit conditions are an important part of the total design, although attention is focused initially on strength.

Historically, members were proportioned so that stresses in the steel and concrete resulting from normal service loads were within specified limits. These limits, known as *allowable stresses*, were only fractions of the failure stresses of the materials. For members proportioned on such a service load basis, the margin of safety was provided by stipulating allowable stresses under service loads that were appropriately small fractions of the compressive concrete strength and the steel yield stress. We now refer to this basis for design as *service load design*. Allowable stresses, in practice, were set at about one-half the concrete compressive strength and one-half the yield stress of the steel.

Because of the difference in realism and reliability, the strength design method has displaced the older service load design method. However, the older method provides the basis for some serviceability checks and is the design basis for many older structures. Throughout this text, strength design is presented almost exclusively.

1.6 DESIGN CODES AND SPECIFICATIONS

The design of concrete structures such as those of Figs. 1.1 to 1.12 is generally done within the framework of codes giving specific requirements for materials, structural analysis, member proportioning, etc. The International Building Code (Ref. 1.2) is an example of a consensus code governing structural design and is often adopted by local municipalities. The responsibility of preparing material-specific portions of the codes rests with various professional groups, trade associations, and technical institutes. In contrast with many other industrialized nations, the United States does not have an official, government-sanctioned, national code.

The American Concrete Institute (ACI) has long been a leader in such efforts. As one part of its activity, the American Concrete Institute has published the widely recognized *Building Code Requirements for Structural Concrete and Commentary* (Ref. 1.12), which serves as a guide in the design and construction of reinforced concrete buildings. The ACI

Code has no official status in itself. However, it is generally regarded as an authoritative statement of current good practice in the field of reinforced concrete. As a result, it has been incorporated by reference into the International Building Code and similar codes that are, in turn, adopted by law into municipal and regional building codes that do have legal status. Its provisions thereby attain, in effect, legal standing. Most reinforced concrete buildings and related construction in the United States are designed in accordance with the current ACI Code. It has also served as a model document for many other countries. The commentary incorporated in Ref. 1.12 provides background material and rationale for the Code provisions. The American Concrete Institute also publishes important journals and standards, as well as recommendations for the analysis and design of special types of concrete structures such as shown in Fig. 1.7.

Most highway bridges in the United States are designed according to the requirements of the AASHTO bridge specifications (Ref. 1.3), which not only contain the provisions relating to loads and load distributions mentioned earlier but also include detailed provisions for the design and construction of concrete bridges. Some of the provisions follow ACI Code provisions closely, although a number of significant differences will be found.

The design of railway bridges is done according to the specifications of the AREMA *Manual of Railway Engineering* (Ref. 1.4). It, too, is patterned after the ACI Code in most respects, but it contains much additional material pertaining to railway structures of all types.

No code or design specification can be construed as a substitute for sound engineering judgment in the design of concrete structures. In structural practice, circumstances are frequently encountered where code provisions can serve only as a guide, and the engineer must rely upon a firm understanding of the basic principles of structural mechanics applied to reinforced or prestressed concrete, and an intimate knowledge of the nature of the materials.

1.7 SAFETY PROVISIONS OF THE ACI CODE

The safety provisions of the ACI Code are given in the form of Eqs. 1.3*b*) and (1.3*c*) using strength reduction factors and load factors. These factors are based on statistical information, experience, engineering judgment, and compromise. In words, the design strength ϕS_n of a structure or member must be at least equal to the required strength U calculated from the factored loads, that is,

$$\text{Design strength} \geq \text{Required strength}$$

or

$$\phi\, S_n \geq U \tag{1.4}$$

The nominal strength S_n is computed (usually somewhat conservatively) by accepted methods. The required strength U is calculated by applying appropriate load factors to the respective service loads: dead load D; live load L; wind load W; earthquake load E; snow load S; rain load R; cumulative effects T due to differential settlement and restrained volume change due to creep, shrinkage, and temperature change; fluid pressure F; and earth pressure H. Loads are defined in a general sense, to include either loads or the related internal effects such as moments, shears, and thrusts. Thus, in specific terms for a member subjected, say, to moment, shear, axial load, and torsional moment

$$\phi M_n \geq M_u \tag{1.5a}$$

$$\phi V_n \geq V_u \tag{1.5b}$$

$$\phi P_n \geq P_u \tag{1.5c}$$

$$\phi T_n \geq T_u \tag{1.5d}$$

where the subscripts n denote the nominal strengths in flexure, shear, and axial load, respectively, and the subscripts u denote the factored load moment, shear, axial load and torsion. In computing the factored load effects on the right, load factors may be applied either to the service loads themselves or to the internal load effects calculated from the service loads.

The load factors specified in the ACI Code, to be applied to calculated dead loads and those live and environmental loads specified in the appropriate codes or standards, are summarized in Table 1.2. A maximum load factor of 1.0 is used for wind load *W* and earthquake load *E* because these loads are expressed at strength level in Ref. 1.1. In addition to the load combinations shown in Table 1.2, Chapter 5 of the ACI Code addresses how load effects due to differential settlement, creep, shrinkage, temperature change, fluid pressure, and earth pressure should be handled depending on the load combination and whether they add or counteract the effects of the primary load. The load combinations in Table 1.2 are consistent with the concepts introduced in Section 1.4 and with ASCE/SEI 7, *Minimum Design Loads and Other Associated Criteria for Buildings and Other Structures* (Ref. 1.1). For individual loads, lower factors are used for loads known with greater certainty, such as dead load, compared with loads of greater variability, such as live loads. Further, for load combinations such as dead plus live loads plus wind forces, reductions are applied to one load or the other that reflect the improbability that an excessively large live load coincides with an unusually high windstorm. The factors also reflect, in a general way, uncertainties with which internal load effects are calculated from external loads in systems as complex as highly indeterminate, inelastic reinforced concrete structures which, in addition, consist of variable-section members (because of tension cracking, discontinuous reinforcement, etc.). Finally, the load factors also distinguish between two situations, particularly when horizontal forces are present in addition to gravity, that is, the Page 19 situation where the effects of all simultaneous loads are additive, as distinct from that in which various load effects counteract one another. For example, wind load produces an overturning moment, and the gravity forces produce a counteracting stabilizing moment.

TABLE 1.2
Factored load combinations for' determining required strength U in the ACI Code

Primary Load[a]	Factored Load or Load Effect U
Basic[b]	$U = 1.2D + 1.6L$
Dead	$U = 1.4D$
Live	$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
Roof, snow, rain[c]	$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5(1.0L \text{ or } 0.5W)$
Wind[c,d]	$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$
	$U = 0.9D + 1.0W$
Earthquake[c,e]	$U = 1.2D + 1.0E + 1.0L + 0.2S$
	$U = 0.9D + 1.0E$

[a] Where the following represent the loads or related internal moments or forces resulting from the listed factors: D = dead load; E = earthquake; L = live load; L_r = roof live load; R = rain; S = snow; and W = wind. In addition to the loads shown in this table, the ACI Code also requires consideration of loads due to F = fluids; H = earth pressure; and T = cumulative effects of differential settlement and restraint of volume change (creep, shrinkage, temperature change).

[b] The "basic" load condition of $U = 1.2D + 1.6L$ reflects the fact that interior members in buildings generally are not subjected to L_r or S or R and that $1.4D$ rarely governs design.

[c] The load factor on live load L in these load combinations may be reduced up to 0.5, except for garages, areas occupied as places of public assembly, and areas where L is greater than 100 psf.

[d] Versions of ASCE/SEI 7 before 2010 provided wind speeds based on service-level design. If service-level winds are used, $1.6W$ should be used for strength design.

[e] The vertical effects of earthquake are additive to the the dead load effects.

In all cases in Table 1.2, the controlling equation is the one that gives the largest factored load effect U.

The strength reduction factors ϕ in the ACI Code are given different values depending on the state of knowledge, that is, the accuracy with which various strengths can be calculated. Thus, the value for bending is higher than that for shear or bearing. Also, ϕ values reflect the probable importance, for the survival of the structure, of the particular member and of the probable quality control achievable. For both these reasons, a lower value is used for columns than for beams. Table 1.3 gives some of the ϕ values specified in Chapter 21 of the ACI Code.

TABLE 1.3
Strength reduction factors in the ACI Code

Strength Condition	Strength Reduction Factor ϕ
Tension-controlled sections[a]	0.90
Compression-controlled sections[b]	
Members with spiral reinforcement	0.75
Other reinforced members	0.65
Shear and torsion	0.75
Bearing on concrete	0.65
Post-tensioned anchorage zones	0.85
Strut-and-tie models[c]	0.75

[a] Chapter 22 discusses reductions in ϕ for pretensioned members where strand embedment is less than the development length.

[b] Chapter 4 contains a discussion of the linear variation of ϕ between tension and compression-controlled sections. Chapter 9 discusses the conditions that allow an increase in ϕ for spirally reinforced columns.

[c] Chapter 17 describes strut-and-tie models.

The joint application of strength reduction factors (Table 1.3) and load factors (Table 1.2) is aimed at producing approximate probabilities of understrength of the order of 1⁄100 and of overloads of 1⁄1000. This results in a probability of structural failure on the order of 1⁄100,000.

1.8 DEVELOPING FACTORED GRAVITY LOADS

To be of use in design, the live loads in Table 1.1 and information on the self-weight of the structural members and other dead loads must be converted into forces acting on the structure. By way of several examples, this section describes the conventions by which this is done for gravity loads.

Figure 1.15 shows a hospital building with a reinforced concrete frame. The masonry fascia and steel entrance give little indication of the underlying structure. Figure 1.16 shows the same building under construction. The slabs, beams, columns, and stairwells are identified. Temporary formwork and shoring for the cast-in-place framing system are visible in the upper stories. This structure is designed for wind and gravity loads because wind loads exceed the earthquake effects at this location. The stairwell walls provide lateral stability and resistance to wind load. The remaining structural elements are designed for gravity loads.

FIGURE 1.15

Hospital building (*Photograph by Charles W. Dolan*)

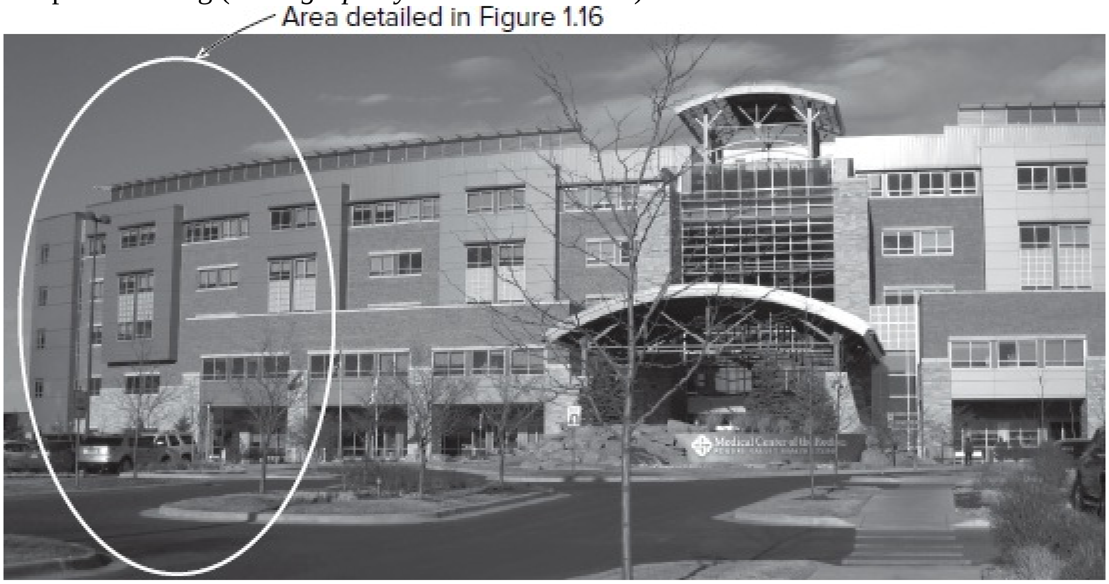


FIGURE 1.16

Details of framing system (*Photograph by Charles W. Dolan*)

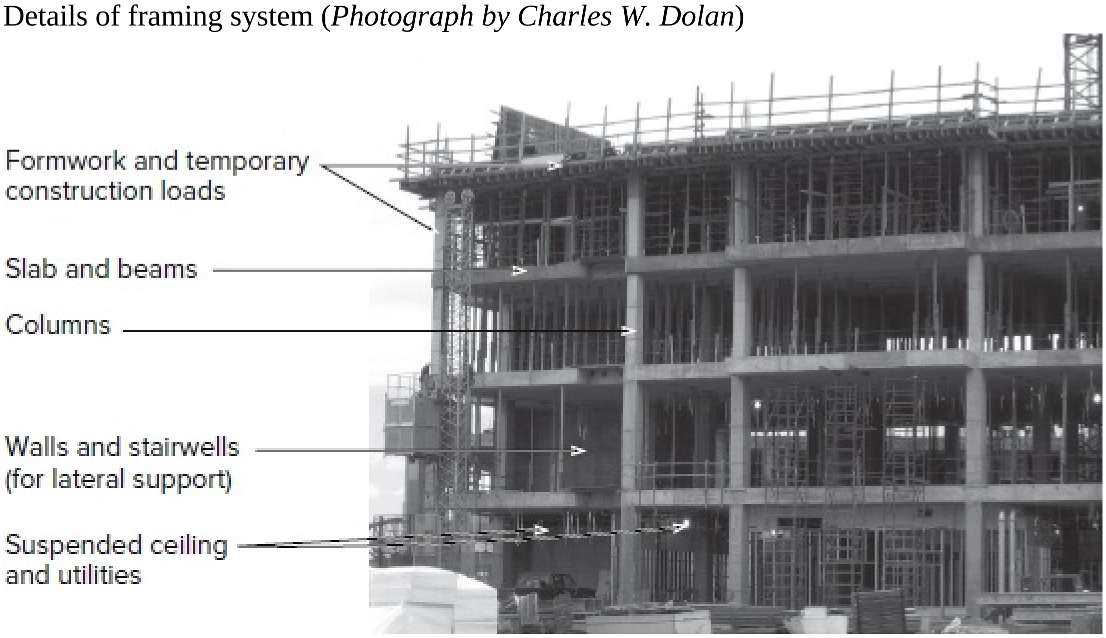
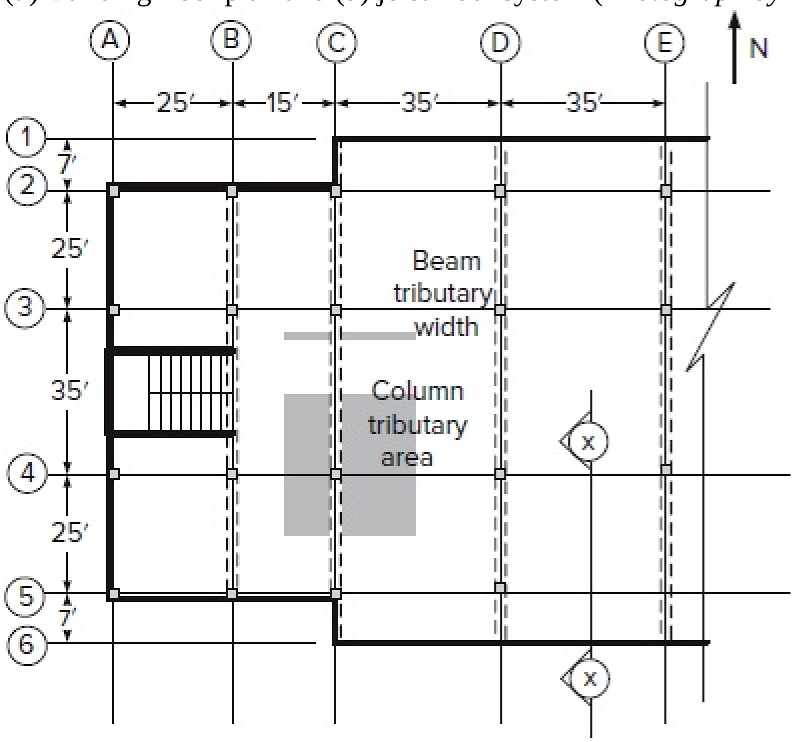


Figure 1.17 shows a schematic floor plan of the building and a photograph of the one-way joist floor system. The slab is 5 in. thick, and the joists (narrow beams not shown in the floor plan) are 6 in. wide, 24 in. deep, spaced at 5 ft, and run in the East–West direction between supporting girders that run North–South between columns. The bays adjacent to building line C are selected to illustrate the development of factored gravity loads to be used

in design. Operating rooms are located in this portion of the building. Preliminary sizing of a typical floor indicates that the girder cross section will be 24 in. deep by 16 in. wide. In addition to the live load, the floor supports a suspended ceiling and duct work below weighing 6.5 psf. Normalweight concrete, producing reinforced concrete with a unit weight of 150 pcf, is used for construction.

FIGURE 1.17

(*a*) Building floor plan and (*b*) joist floor system (*Photograph by Charles W. Dolan*)



(*a*) Floor plan

(*b*) Section X-X joist floor system

EXAMPLE 1.1 **Loads on slab ⁄ joist system.** Determine the service and factored loads acting on the slab ⁄ joist system between lines C and D.

SOLUTION. Slab loads are typically defined as surface loads in pounds per square foot (psf). Page 21 From Table 1.1, the live load for hospital operating rooms is 60 psf. The slab is 5 in. thick. The joists are 6 in. wide by 24 in. deep, extending 19 in. below the bottom of the slab and spaced 5 ft on center. The joists can be considered as adding to the thickness of the slab for the purpose of calculating the dead load of the system. The equivalent increase in slab thickness equals the cross-sectional area of the joist below the bottom of the slab divided by the spacing of the joists in in. or (6 × 19) ⁄ (5 × 12) = 1.9 in., giving an equivalent total slab thickness for calculation of dead load of 6.9 in. The dead load of the slab ⁄ joist system is then the equivalent total slab thickness in feet times the concrete density, 150 pcf, resulting in a slab dead weight of (6.9 ⁄ 12) × 150 pcf = 86.3 psf. The service load q_s on the slab is then 86.3 psf + 6.5 psf superimposed dead load + 60 psf live load, giving q_s = 152.8 psf. The factored load for this example is determined using the basic load factor condition from Table 1.2. Thus, the factored load on the slab q_u is 1.2 × (86.3 psf + 6.5 psf) + 1.6 × 60 psf = 207.4 psf, which is rounded to q_u = 207 psf.

EXAMPLE 1.2 **Load on girder.** Determine the factored load applied to the interior girder on line C between lines 3 and 4.

SOLUTION. Beam and girder loads are Page 22

typically defined in pounds per linear foot (plf) or kips per linear foot (klf) along the length of the beam. The loads are developed using a 1 ft wide tributary strip perpendicular to the girder, shown in Fig. 1.17*a*. In this example, the length of the tributary strip goes halfway across the slabs loading the girder and is, thus, 7.5 ft long on the B-C side and 17.5 ft long on the C-D side of building line C for a total length of 25 feet. From Example 1.1, the factored load on the slab, 207 psf, is applied to the 1 ft wide strip, giving a load on the girder of 207 psf × 1 ft wide × 25 ft total tributary width = 5175 plf. To this must be added the factored girder self-weight. Only the 19 in. deep portion below the slab need be added, thus the load must be increased by 16 in. wide × 19 in. deep × 150 pcf ⁄ 144 in^2/ft^2 = 317 plf. The uniform factored design load on the girder w_u is then 5175 plf + 1.2 × 317 plf = 5555.4 plf. Using three significant figures, the load to be used in design is w_u = 5.56 kip ⁄ ft.

EXAMPLE 1.3 **Load on column.** Determine the factored axial load transferred to column C4.

SOLUTION. Column axial loads are expressed in pounds or kips and are established using a tributary area. The tributary area for column C4, shown in Fig. 1.17*a*, is a rectangular area measure halfway between column lines (that is, half the distance to the adjacent columns) equal to (15 ft ⁄ 2 + 35 ft ⁄ 2) × (25 ft ⁄ 2 + 35 ft ⁄ 2) = 25 × 30 = 750 ft^2. From Example 1.1, the factored load of the slab is q_u = 207 psf to which must be added the factored weight of the beam from Example 2, 1.2 × 317 plf, within the tributary area. Thus, the load transferred to the column is P_u = 207 psf

× 750 ft^2 + 1.2 × 317 plf × (35 ft ⁄ 2 + 25 ft ⁄ 2) = 166,650 lb or 166.7 kips. Using three significant figures, the factored axial load would be P_u = 167 kips. The determination of axial loads for use in design is discussed further in Section 11.3.

1.9 CONTRACT DOCUMENTS AND INSPECTION

Design information is transmitted from the *licensed design professional*, sometimes called the engineer of record, to the contractor through the *contract documents*. These documents typically consist of *plans*, *specifications*, and *estimates*. A typical set of plans includes the graphical information describing the architectural, structural, mechanical, and electrical components of the building. The structural plans contain the concrete sections, reinforcement, reinforcement details and placement, and other technical information based on the engineer's calculations. The licensed design professional commonly includes sketches or drawings of specific details in the calculations to allow the information to be accurately incorporated into the plans.

Specifications consist of two parts: the contract terms and conditions and the technical specifications. Contract terms and conditions include what is to be constructed, the time and cost of the construction, bonding requirements, and other specific issues between the owner and the contractor. Technical specifications contain the detailed information the contractor needs to complete a project and are provided by the licensed design professional. They include the ASTM specifications for concrete and steel, requirements for concrete strength and placement, grade of reinforcement to be used, considerations for hot and cold weather concreting, and other project specific information. During design, the licensed design professional makes decisions about the materials and details needed to comply with the design Page 23 intent and the building code requirements. A contractor is not required to be familiar with the ACI Building Code nor is the contractor responsible for assuring that code requirements are satisfied. Therefore, the licensed design professional must include all relevant project and code requirements in the plans and specifications.Chapter 26 of the ACI Building Code (Ref. 1.12) contains a comprehensive description of information that the engineer must include in the plans and specifications. Pointers to this chapter are included throughout the ACI Code to assist the licensed design professional in finding and recording the correct information.

Inclusion of information in the project specifications does not, by itself, assure that construction will be executed according to the design intent. Rather, compliance requirements and inspection provide the licensed design professional and the owner with confirmation that the design intent is being met. Compliance requirements associated with project specifications are provided in Chapter 26 of the ACI Code. These compliance requirements complement the technical specifications by providing direct feedback to the licensed design professional. For example, if the specifications require the concrete strength to be 6000 psi, the compliance requirement would be that the strength test results, based on ASTM specifications for testing concrete, demonstrate the concrete meets or exceeds the specified strength. Actual testing of materials is done by testing agencies and technicians certified by the American Concrete Institute or other qualification agencies.

Inspection further advances compliance with the design intent. Inspection can range from onsite observations to detailed investigation of reinforcement placement. Onsite observations are conducted intermittently as a general overview of the construction with the intent of confirming overall design intent. Such observations may lead to discussions with the contractor regarding the way the work is done but do not direct the contractor’s work. In areas prone to earthquakes, special inspection may be required. Special inspection requires the licensed design professional or a certified designee to conduct the inspections. These inspections specifically examine those elements of the design required to resist earthquake load effects and to certify that the construction meets the design details. The General Building Code and the ACI Building Code specify situations where special inspection is required.

The licensed design professional is sometimes required to provide an estimate of the cost of construction. This estimate addresses several issues. The estimate initially provides the owner with information to indicate that the available project funding is adequate. It can also provide a basis for estimating the degree of completion during construction. Cost estimates solely by the engineer are most often associated with engineered projects, such as bridges, piers, and industrial facilities. Cost estimates for building construction are typically provided by the architect with input from the engineer.

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PROBLEMS

All problems refer to Table 1.1 for live loads and Fig. P1.1 for the building layout. No live load reduction factors are considered. Figure P1.1 provides a plan and elevation of a reinforced concrete building 7 bays long by 4 bays wide. The building has beams along building lines A through E and one-way slabs spanning between building lines A through E. A central stairwell ⁄ elevator shaft between building lines 5 and 6 provides the lateral support, so only gravity loads need to be calculated. The bay dimensions, beam dimensions, and occupancy uses are given in the individual problem statements below. Construction is with normalweight concrete with a density of 150 pcf for the purposes of calculating dead load.

1.1. The building in Fig. P1.1 is used for general office space. The slab is 8 in. thick. The beams are 12 in. wide and have a total depth of 18 in., the bay dimensions are 18.5 ft in the X direction and 21 ft in the Y direction, and the superimposed service dead load is 25 psf. Calculate the slab service load in psf and the interior beam service load in klf. (**Solution:** q_s = 175 psf, w_s = 3.36 klf.)

1.2. The building in Fig. P1.1 is used for general office space. The slab is 8 in. thick. The beams are 12 in. wide and have a total depth of 18 in., the bay dimensions are 18.5 ft in the X direction and 21 ft in the Y direction, and the superimposed service dead load is 25 psf. Calculate the factored axial column load transferred to column C3 on the third floor. (**Solution:** P_u = 92.5 kips.)

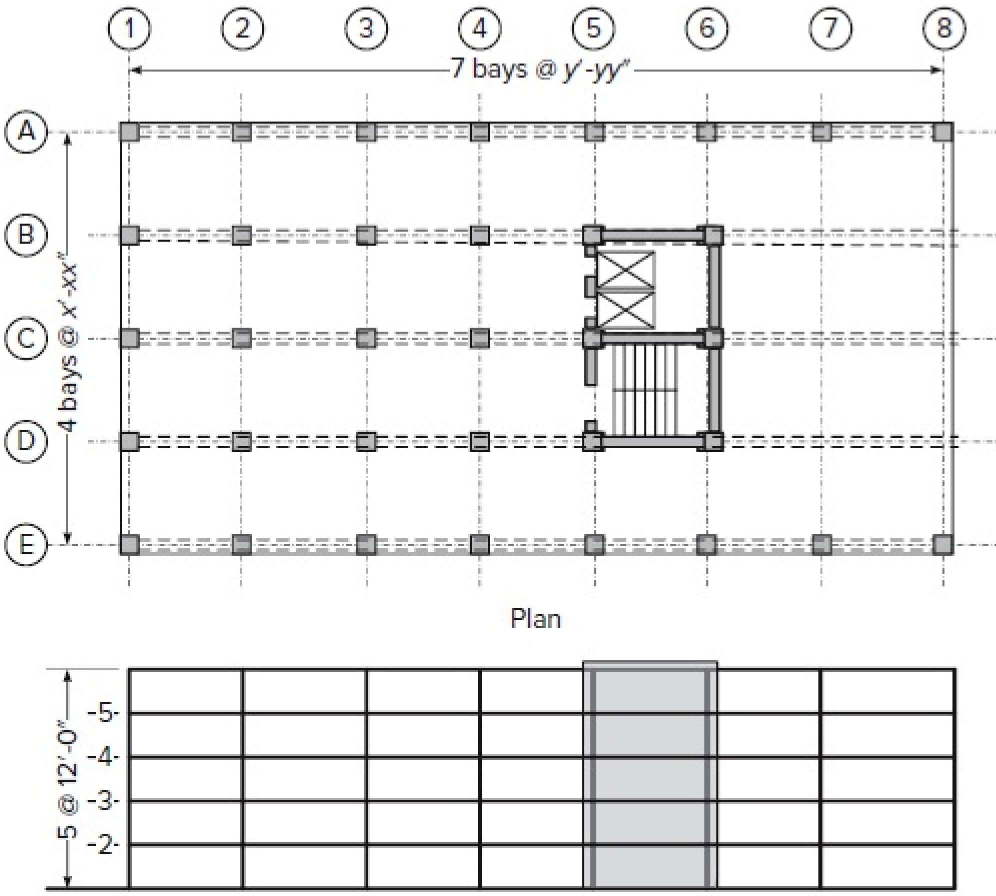
1.3. The building in Fig. P1.1 is used for general office space. The slab is

8 in. thick. The beams are 12 in. wide and have a total depth of 18 in., the bay dimensions are 18.5 ft in the *X* direction and 21 ft in the *Y* direction, and the superimposed service dead load is 25 psf. Calculate the slab factored load in psf and the beam factored load in klf. Comment on your solution in comparison with Problem 1.1.

1.4. A slab in Fig. P1.1 is used for lobby space. The slab is 10 in. thick. The beams are 14 in. wide and have a total depth of 24 in., the bay dimensions are 21 ft in the *X* direction and 26 ft in the *Y* direction, and the superimposed service dead load is 15 psf. Calculate the slab factored load in psf and the beam factored load in klf.

1.5. The building in Fig. P1.1 is used for light storage space. The slab is 10 in. thick. The beams are 16 in. wide and have a total depth of 20 in., the bay dimensions are 20 ft in the *X* direction and 25 ft in the *Y* direction, and the superimposed sprinkler dead load is 4 psf. Calculate the slab factored load in psf and the beam factored load in klf.

FIGURE P1.1
Building plan and elevation



Elevation

1.6. The roof on the building in Fig. P1.1 has a slab that is 7 in. Page 25 thick. The beams are 12 in. wide and have a total depth of 16 in., the bay dimensions are 19 ft in the X direction and 21 ft in the Y direction, and the superimposed service dead load is 6 psf. Calculate the slab factored load in psf and the beam factored load in klf given the roof snow load is 30 psf.

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† Abbreviation for kips per square inch, or thousands of pounds per square inch.

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† Throughout this book quantities that refer to the strength of members, calculated by accepted analysis methods, are furnished with the subscript n, which stands for “nominal.” This notation is in agreement with the ACI Code. It is intended to convey that the actual strength of any member is bound to deviate to some extent from its calculated, nominal value because of inevitable variations of dimensions, materials properties, and other parameters. Design in all cases is based on this nominal strength, which represents the best available estimate of the actual member strength.

2 Materials

2.1 INTRODUCTION

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

2.2 CEMENT

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

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

Over the years, five standard types of portland cement have been developed. Type I, *normal* portland cement, is used for over 90 percent of construction in the United States. Concretes made with Type I portland cement generally need one to two weeks to reach sufficient strength so that forms of beams and slabs can be removed and reasonable loads Page 27 applied; they reach their design strength after 28 days and continue to gain strength thereafter at a decreasing rate. To speed construction when needed, *high early strength cements* such as Type III have been developed. They are costlier than ordinary portland cement, but within 7 to 14 days they reach the strength achieved using Type I at 28 days. Type III portland cement contains the same basic compounds as Type I, but the relative proportions differ and it is ground more finely.

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

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

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

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

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

2.3 AGGREGATES

In ordinary structural concretes the aggregates occupy 65 to 75 percent of the volume of the hardened mass. The remainder consists of hardened cement paste, uncombined water (that is, water not involved in the hydration of the cement), and air voids. The latter two do not contribute to the strength of the concrete. In general, the more densely the aggregate can be packed, the better the durability and economy of the concrete. For this reason the gradation of the particle sizes in the aggregate, to produce close packing, is important. It is also important that the aggregate have good strength, durability, and weather resistance; that its surface be free from impurities such as loam, clay, silt, and organic matter that may weaken the bond with cement paste; and that no unfavorable chemical reaction take place between it and the cement.

Natural aggregates are generally classified as fine and coarse. *Fine aggregate* (typically natural sand) is any material that will pass a No. 4 sieve, that is, a sieve with four openings per linear inch. Material coarser than this is classified as *coarse aggregate*. When favorable gradation is desired, aggregates are separated by screening into two or three size groups of sand and several size groups of coarse aggregate. These can then be combined according to grading criteria to provide a densely packed aggregate. The *maximum size of coarse aggregate* in reinforced concrete is governed by the requirement that it must easily fit into the forms and between the reinforcing bars. For this purpose it should not be larger than one-fifth of the narrowest dimension of the forms or one-third of the depth of slabs, nor three-quarters of the minimum distance between reinforcing bars. Requirements for satisfactory aggregates are found in ASTM C33, "Standard Specification for Concrete Aggregates," and authoritative information on aggregate properties and their influence on concrete properties, as well as guidance in selection, preparation, and handling of aggregate, is found in Refs. 2.1 and 2.2.

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

A variety of *lightweight* aggregates are available. Some unprocessed aggregates, such as pumice or cinders, are suitable for insulating concretes, but for structural lightweight concrete, *processed aggregates* are used because of better control. These consist of expanded shales, clays, slates, slags, or pelletized fly ash. They are light in weight because of the porous, cellular structure of the individual aggregate particles, which is achieved by gas or steam formation in processing the aggregates in rotary kilns at high temperatures (generally in excess of 2000°F). Requirements for satisfactory lightweight aggregates are found in ASTM C330, "Standard Specification for Lightweight Aggregates for Structural Concrete."

Structural lightweight concretes have unit weights between 70 and 120 pcf, with most in the range of 105 to 120 pcf. Lower density lightweight concretes typically have compressive strengths of 1000 to 2500 psi and are chiefly used as fill, such as over light-gage steel floor panels. Lightweight concretes with unit weights between 90 and 120 pcf have compressive strengths comparable to those of normalweight concretes. Similarities and differences in structural characteristics of lightweight and normalweight concretes are discussed in Sections 2.8 and 2.9.

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

2.4 PROPORTIONING AND MIXING CONCRETE

Page 29

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

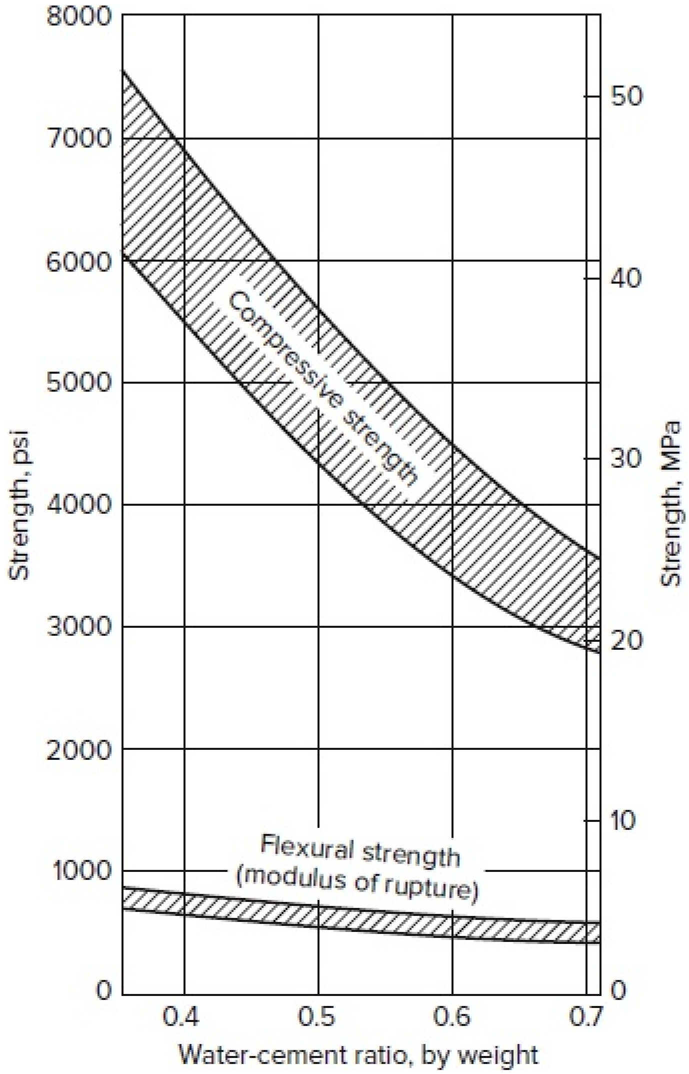
Figure 2.1 shows the decisive influence of the water-cement ratio on the compressive strength of concrete. Its influence on tensile strength, as measured by the nominal flexural strength or modulus of rupture, is also seen to be pronounced but much less than its effect on compressive strength. This seems to be so because, in addition to the void ratio, the tensile

Page 30

strength depends on the strength of the bond between coarse aggregate and mortar (that is, cement paste plus fine aggregate). Tests show that this bond strength is only slightly affected by the water-cement ratio (Ref. 2.4).

FIGURE 2.1

Effect of water-cement ratio on 28-day compressive and flexural tensile strength. (*Adapted from Ref. 2.3.*)



It is customary to define the *proportions* of a concrete mix in terms of the total weight of each component needed to make up 1 yd^3 of concrete, such as 517 lb of cement, 300 lb of water, 1270 lb of sand, and 1940 lb of coarse aggregate, plus the total volume of air, in percent. Air content is typically 4 to 7 percent when air is deliberately *entrained* in the mix and 1 to 2 percent when it is not. The weights of the fine and coarse aggregates are based on material in the *saturated surface dry condition*, in which, as the description implies, the aggregates are fully saturated but have no water on the exterior of the particles.

Various methods of proportioning are used to obtain mixes of the desired properties from the cements and aggregates at hand. One is the *trial-batch method*. Selecting a water-cement ratio from information such as that in Fig.

2.1, one produces several small trial batches with varying amounts of aggregate to obtain the required strength, consistency, and other properties with a minimum amount of paste. Concrete *consistency* is most frequently measured by the *slump test*. A metal mold in the shape of a truncated cone 12 in. high is filled with fresh concrete in a carefully specified manner. Immediately upon being filled, the mold is lifted off, and the slump of the concrete is measured as the difference in height between the mold and the pile of concrete. The slump is a good measure of the total water content in the mix and should be kept as low as is compatible with workability. Slumps for concretes in building construction generally range from 2 to 5 in., although higher slumps are used with the aid of chemical admixtures, especially when very fluid mixtures are needed to allow the concrete to be placed between closely spaced reinforcing bars.

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

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

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

The principal purpose of *mixing* is to produce an intimate mixture of cement, water, fine and coarse aggregate, and possible admixtures of uniform consistency throughout each batch. This is typically achieved in machine mixers of the revolving- drum type. Minimum mixing time is 1 min for mixers of not more than 1 yd^3 capacity, with an additional 15 sec for each additional 1 yd^3. Mixing can be continued for a considerable time without adverse effect. This fact is particularly important in connection with Page 31 ready mixed concrete.

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

Much information on proportioning and other aspects of design and control of concrete mixtures will be found in Refs. 2.7 and 2.8.

2.5 CONVEYING, PLACING, CONSOLIDATING, AND CURING

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

Placing is the process of transferring the fresh concrete from the conveying device to its final place in the forms. Prior to placing, loose rust must be removed from reinforcement, forms must be cleaned, and hardened surfaces of previous concrete lifts must be cleaned and treated appropriately. Placing and consolidating are critical in their effect on the final quality of the concrete. Proper placement must avoid segregation, displacement of forms or of reinforcement in the forms, and poor bond between successive layers of concrete. Immediately upon placing, the concrete should be *consolidated,* usually by means of vibrators. Consolidation prevents honeycombing, ensures close contact with forms and reinforcement, and serves as a partial remedy to possible prior segregation. Consolidation is achieved by high-frequency, power-driven *vibrators*. These are of the *internal* type, immersed in the concrete, or of the *external* type, attached to the forms. The former are preferable but must be supplemented by the latter where narrow forms or other obstacles make immersion impossible (Ref. 2.9). Vibration is not needed for *self-consolidating concrete*, a fluid concrete that consolidates under its own weight, discussed in more detail in Section 2.7.

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

To prevent such damage, concrete should be protected from loss of moisture for at least 7 days and, in more sensitive work, up to 14 days. When high early strength cements are used, curing periods can be cut in half. Curing can be achieved by keeping exposed surfaces continually wet through sprinkling, ponding, or covering with plastic film or by the use of sealing compounds, which, when properly used, form evaporation-retarding membranes. In addition to improving strength, proper moist-curing provides better shrinkage control. To protect concrete against low temperatures during cold weather, the mixing water, and occasionally the aggregates, is heated; thermal insulation is used where possible; and special admixtures are employed. When air temperatures are very low, external heat may have to be supplied in addition to insulation (Refs. 2.7, 2.8, 2.10, and 2.11).

2.6 QUALITY CONTROL

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

The main measure of the structural quality of concrete is its *compressive strength*. Tests for this property are made on cylindrical specimens of height equal to twice the diameter, usually 6 × 12 in. or 4 × 8 in. Impervious molds of this shape are filled with concrete during construction as specified by ASTM C172, "Standard Method of Sampling Freshly Mixed Concrete," and ASTM C31, "Standard Practice for Making and Curing Concrete Test Specimens in the Field." The cylinders are moist-cured at about 70°F, generally for 28 days, and then tested in the laboratory at a specified rate of loading. The compressive strength obtained from such tests is known as the *cylinder strength*, which is compared to the *specified compressive strength* f_c', the main property specified for design.

To provide structural safety, continuous control is necessary to ensure that the strength of the concrete as furnished is in satisfactory agreement with the value called for by the designer. The ACI Code specifies that at least two 6 × 12 in. or three 4 × 8 in. cylinders must be tested for each 150 yd^3 of concrete or for each 5000 ft^2 of surface area actually placed, but not less than once a day. As mentioned in Section 2.4, the results of strength tests of different batches mixed to identical proportions show inevitable scatter. The scatter can be reduced by closer control, but occasional tests below the cylinder strength specified in the design cannot be avoided.

To ensure adequate concrete strength in spite of such scatter, the ACI Code stipulates that concrete quality is satisfactory if

1. Every average of any three consecutive strength tests equals or exceeds f_c' , and

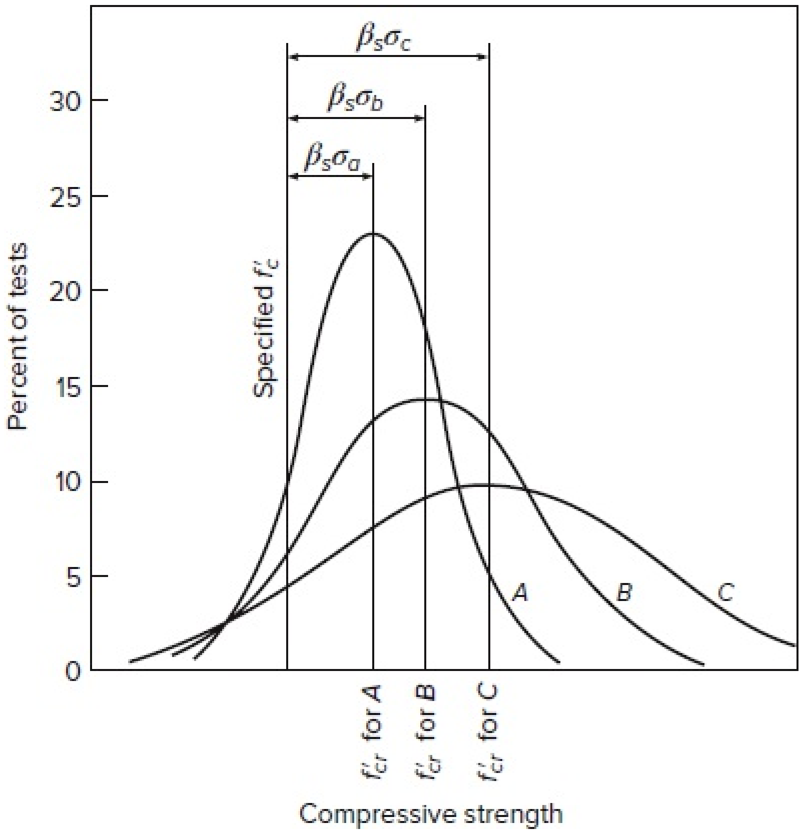
2. No strength test (the average of two or three cylinder tests depending on cylinder size) falls below the required f_c' by more than 500 psi if f_c' is 5000 psi or less or by more than 0.10 f_c' if f_c' exceeds 5000 psi.

It is evident that if concrete were proportioned so that its mean strength were just equal to the required strength f_c' , it would not pass these quality requirements, because about one-half of the strength test results would fall below the required f_c' . It is therefore necessary to proportion the concrete so that its mean strength , used as the basis for selection of suitable Page 33 proportions, exceeds the specified design strength f_c' by an amount sufficient to ensure that the two quoted requirements are met. The minimum amount by which the required mean strength must exceed f_c' can be determined only by statistical methods because of the random nature of test scatter. Requirements have been derived, based on statistical analysis, to be used as a guide to proper proportioning of the concrete at the plant so that the probability of strength deficiency at the construction site is acceptably low.

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

FIGURE 2.2

Frequency curves and average strengths for various degrees of control of concretes with specified design strength f_c' . (*Adapted from Ref. 2.12.*)



On the basis of such studies, ACI Code 26.4.3.1 requires that mixture proportions be established in accordance with ACI 301, “Specifications for Structural Concrete” (Ref. 2.13). ACI 301 requires concrete production facilities to maintain records from which the standard deviation achieved in the particular facility can be determined. ACI 301 also stipulates the minimum amount by which the required average compressive strength , aimed at when selecting concrete proportions, must exceed the specified compressive strength f_c' . In accordance with ACI 301, the value of is equal to the larger of the values in Eqs. (2.1) and (2.2).

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

or

$$f'_{cr} = \begin{cases} f'_c + 2.33ks_s - 500 & \text{for } f'_c \le 5000 \text{ psi} \\ 0.9f'_c + 2.33ks_s & \text{for } f'_c > 5000 \text{ psi} \end{cases} \qquad (2.2a)(2.2b)$$

where s_s is the standard deviation of the test sample. The value of k is given in Table 2.1.

TABLE 2.1
Modification factor k for sample standard deviation s_s when less than 30 tests are available

No. of Tests†	Modification Factor k for Sample Standard Deviation
Less than 15	See paragraph following Eqs. (2.1) and (2.2)
15	1.16
20	1.08
25	1.03
30 or more	1.00

† Interpolate for intermediate values.

Equation (2.1) provides a probability of 1 in 100 that averages of three consecutive tests will be below the specified strength f_c' . Equations (2.2*a*) and (2.2*b*) provide a probability of 1 in 100 that an individual strength test will be more than 500 psi below the specified f_c' for f_c' up to 5000 psi or below $0.90f_c'$ for f_c' over 5000 psi.

To use Eqs. (2.1) and (2.2), ACI 301 (Ref. 2.13) requires that a minimum of 15 consecutive test results be available. The tests must represent concrete with (1) a specified compressive strength within 1000 psi of f_c' for the project and (2) materials, quality control, and conditions similar to those expected for the building in question. If fewer than 15 tests have been made, must exceed f_c' + 1000 psi for f_c' less than or equal to 3000 psi, f_c' + 1200 psi for f_c' between 3000 and 5000 psi, and 0.1 f_c' + 700 psi for f_c' over 5000 psi.

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

Both the requirements described earlier in this section for determining if concrete, as produced, is of satisfactory quality and the process just described of selecting are based on the same basic considerations but are applied independently, as demonstrated in Examples 2.1 and 2.2.

EXAMPLE 2.1 **Average required strength.** A building design calls for specified concrete strength f_c' of 4000 psi.

Page 35

Calculate the average required strength if (*a*) 30 consecutive tests for concrete with similar strength and materials produce a sample standard deviation s_s of 535 psi, (*b*) 15 consecutive tests for concrete with similar strength and materials produce a sample standard deviation s_s of 510 psi, and (*c*) less than 15 tests are available.

SOLUTION.

(*a*) 30 tests available. Using s_s = 535 psi and k = 1.0 (from Table 2.1), Eq. (2.1) gives

$$= f_c' + 1.34ks_s = 4000 + 1.34 \times 1.0 \times 535 = 4720 \text{ psi}$$[†]

Because the specified strength f_c' is less than 5000 psi, Eq. (2.2*a*) must be used.

$$= f_c' + 2.33ks_s - 500 = 4000 + 2.33 \times 1.0 \times 535 - 500 = 4750 \text{ psi}$$

The required average strength is equal to the larger value, 4750 psi.

(*b*) 15 tests available. Because only 15 tests are available, s_s the factor $k = 1.16$ from Table 2.1.

$$1.16 \times s_s = 1.16 \times 510 = 590 \text{ psi}$$

Using $s_s = 510$ and $k = 1.16$, Eqs. (2.1) and (2.2*a*) give, respectively,

$$= 4000 + 1.34 \times 1.16 \times 510 = 4790 \text{ psi}$$

$$= 4000 + 2.33 \times 1.16 \times 510 - 500 = 4880 \text{ psi}$$

The larger value, 4880 psi, is selected as the required average strength .

(*c*) Less than 15 tests available. Because f_c' is between 3000 and 5000 psi, the required average strength is

$$= f_c' + 1200 = 4000 + 1200 = 5200 \text{ psi}$$

This example demonstrates that in cases where test data are available, good quality control, represented by a low sample standard deviation s_s, can be used to reduce the required average strength . The example also demonstrates that a lack of certainty in the value of the standard deviation due to the limited availability of data results in higher values for , as shown in parts (*b*) and (*c*). As additional test results become available, the higher safety margins can be reduced.

EXAMPLE 2.2 **Satisfactory test results.** The first eight compressive strength test results for the building described in Example 2.1*c* are 4730, 4280, 3940, 4370, 5180, 4870, 4930, and 4850 psi.

(*a*) Are the test results satisfactory, and (*b*) in what fashion, if any, should the mixture proportions of the concrete be altered?

SOLUTION.

(*a*) For concrete to be considered satisfactory, every arithmetic mean of any three consecutive tests must equal or exceed f_c' , and no individual test may fall below $f_c' - 500$ psi. The eight tests meet these criteria. The average of all sets of three consecutive tests exceeds f_c' [for example, (4730 + 4280 + 3940) ⁄ 3 = 4320, (4280 + 3940 + 4370) ⁄ 3 = 4200, etc.], and no test is less than $f_c' - 500$ psi = 4000 − 500 = 3500 psi.

(*b*) To determine if the mixture proportions must be altered, we note that the solution to Example 2.1*c* requires that equal or exceed 5200 psi. The average of the first eight tests is 4640 psi, well below the value of . Thus, the mixture proportions should be modified by decreasing the water-cement ratio to increase the concrete strength. Once at least 15 tests are available, the value of can be recalculated using Eqs. (2.1) and (2.2) with the appropriate factor *k* from Table 2.1. The mixture proportions can then be adjusted based on the new value of , the strength of the concrete being produced, and the level of quality control, as represented by the sample standard deviation s_s.

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

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

2.7 ADMIXTURES

In addition to the main components of concretes, *admixtures* are often used to improve concrete performance. There are admixtures to accelerate or retard setting and hardening, improve workability, increase strength, improve durability, decrease permeability, and impart other properties (Ref. 2.14). The beneficial effects of particular admixtures are well established. Chemical admixtures should meet the requirements of ASTM C494, "Standard Specification for Chemical Admixtures for Concrete."

Air-entraining agents are widely used. They cause the formation of small dispersed air bubbles in the concrete. These improve workability and durability (chiefly resistance to freezing and thawing) and reduce segregation during placing. They decrease concrete density because of the increased void ratio and thereby decrease strength; however, this decrease can be largely offset by a reduction of mixing water without loss of workability. The chief use of air-entrained concretes is in pavements and structures exposed to the elements (Ref. 2.7).

Accelerating admixtures are used to reduce setting time and accelerate early strength development. Calcium chloride is the most widely used accelerator because of its cost effectiveness, but it should not be used in prestressed concrete and should be used with caution in reinforced concrete in a moist environment, because of its tendency to promote corrosion of steel, or in architectural concrete, because of its tendency to discolor concrete. Nonchloride, noncorrosive accelerating admixtures are available, the principal one being calcium nitrite (Ref. 2.14).

Set-retarding admixtures are used primarily to offset the accelerating