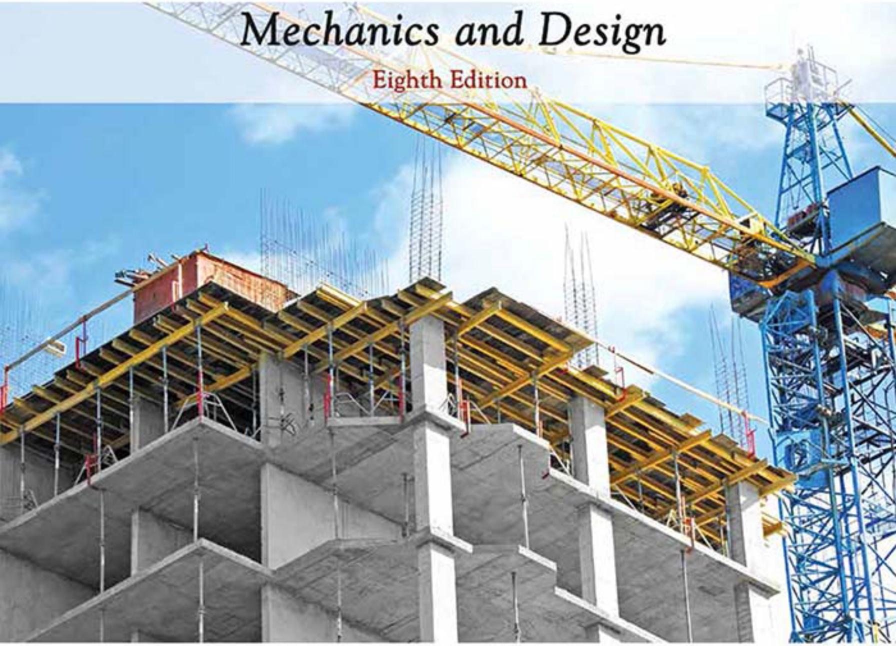
REINFORCED CONCRETE

Mechanics and Design

Eighth Edition



JAMES K. WIGHT

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EIGHTH EDITION

JAMES K. WIGHT

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Preface

Reinforced concrete design encompasses both the art and science of engineering. This book presents the theory of reinforced concrete design as a direct application of the laws of statics and mechanics of materials. It emphasizes that a successful design not only satisfies design rules, but is capable of being built in a timely fashion for a reasonable cost and should provide a long service life.

Philosophy of Reinforced Concrete: Mechanics and Design

A multitiered approach makes *Reinforced Concrete: Mechanics and Design* an outstanding textbook for a variety of university courses on reinforced concrete design. Topics are normally introduced at a fundamental level, and then move to higher levels where prior educational experience and the development of *engineering judgment* will be required. The analysis of the flexural strength of beam sections is presented in Chapter 4. Because this is the first significant design-related topic, it is presented at a level appropriate for new students. Closely related material on the analysis of column sections for combined axial load and bending is presented in Chapter 11 at a somewhat higher level, but still at a level suitable for a first course on reinforced concrete design. Advanced subjects are also presented in the same chapters at levels suitable for advanced undergraduate or graduate students. These topics include, for example, the complete moment versus curvature behavior of a beam section with various tension reinforcement percentages and the use of strain-compatibility to analyze either over-reinforced beam sections, or column sections with multiple layers of reinforcement. More advanced topics are covered in the later chapters, making this textbook valuable for both undergraduate and graduate courses, as well as serving as a key reference in design offices. Other features include the following:

1. Extensive figures are used to illustrate aspects of reinforced concrete member behavior and the design process.

2. Emphasis is placed on logical order and completeness for the many design examples presented in the book.

3. Guidance is given in the text and in examples to help students develop the engineering judgment required to become a successful designer of reinforced concrete structures.

4. Chapters 2 and 3 present general information on various topics related to structural design and construction, and concrete material properties. Frequent References are made back to these topics throughout the text.

Overview

The ACI 318-19 Building Code Requirements for Structural Concrete addresses design and detailing requirements for concrete members and structures. The 2019 Edition of the ACI Code includes several technical updates and revisions from the 2014 Edition of the ACI Code. One of the primary modifications was made to the shear strength equations to include the effects of member depth, low flexural reinforcement ratio, and axial load for one-way beams and slabs and for two-way slabs. By directly addressing these issues, many of the former empirical equations were removed from the ACI Code. The initial goal of these changes was to address both non-prestressed and prestressed members, but the new ACI Code equations (given in Chapter 6) are to be used only for non-prestressed members. Also, for members that include the minimum required amount of transverse (shear) reinforcement and have low or no axial load, the new shear strength equations offer an option that is essentially equivalent to the former shear strength equations used in the ACI Code over the last several decades.

The ACI Code now permits the use of higher-strength reinforcement, specifically Grades 80 and 100. The use of these higher grades of reinforcement impact several ACI Code requirements, but their most significant impact is in the development length requirements given in Chapter 8. Changes made to the development length equations for hooked and headed bars include new requirements for confinement reinforcement to maintain reasonable development lengths for those bars.

Changes and Features for the Eighth Edition

All chapters of the text have been reviewed and updated to be in compliance with the 2019 edition of the ACI Building Code. New problems were developed for several chapters and all of the examples given in the book were either reworked or checked for accuracy. Other changes and some continuing features include the following.

1. The presentation of technical information in Chapters 6, 7, and 17 was rearranged to provide a better flow from discussions of member behavior to development of design code requirements. In Chapter 7, additional information was given for the equivalent tube analogies used to define member strength and behavior before and after torsional cracking.

2. Changes were made in the earthquake-resistant design requirements in Chapter 19 to be in compliance with updates to seismic provisions in the ACI Building Code. Key changes were made for transverse reinforcement requirements at the edges (boundary elements) of special structural walls, the minimum required width of boundary elements, and the calculated deformation capacity of those walls.

3. Flexural design procedures for the full spectrum of beam sections and material strengths are developed in Chapter 5. Although these design procedures are developed for beam sections, they are easily applied to the flexural design of one-way and two-way slab sections.

4. The design of coupled shear walls and coupling beams in seismic regions is given in Chapter 19, including a discussion of coupling beams with moderate span-to-depth ratios, a topic not well-covered in the ACI Building Code.

5. Chapter 2 contains a discussion of sustainability for design and construction of concrete structures. The use of concrete in building construction for reduced CO_2 emissions and life-cycle costs, as well as improved thermal properties and building aesthetics are discussed.

6. Information is provided for structural analysis of both one-way (Chapter 5) and two-way (Chapter 13) continuous floor systems. Typical modeling assumptions for both types of systems and the interplay between analysis and design are discussed.

7. Axial load vs. moment interaction diagrams are given for a variety of column sections in Appendix A. These diagrams include the required strength-reduction factor, and thus, are very useful for section design either in a classroom or a design office.

Use of Textbook in Undergraduate and Graduate Courses

The following paragraphs give a suggested set of topics and chapters to be covered in the first and second reinforced concrete design courses, normally given at the undergraduate and graduate levels, respectively. It is assumed that these are semester courses.

First Design Course

Chapters 1 through 3 should be assigned, but the detailed information on loading in Chapter 2 can be covered in a second course. The information on concrete material properties in Chapter 3 could be covered with more depth in a separate undergraduate materials course. **Chapters 4 and 5** are extremely important for all students and should form the foundation of the first undergraduate course. The information in Chapter 4 on moment vs. curvature behavior of beam sections is important for all designers, but this topic could be significantly expanded in a graduate course. Chapter 5 presents a variety of design procedures for developing efficient flexural designs of either singly-reinforced or doubly-reinforced sections. The discussion of structural analysis for continuous floor systems in Section 5-2 could be skipped if either time is limited or students are not yet prepared to handle this topic. The first undergraduate course should cover **Chapter 6** information on member behavior in shear and the shear design requirements given in the ACI Code. Discussions of other methods for determining the shear strength of concrete members can be saved for a second design course. Design for torsion, as covered in **Chapter 7**, could be covered in a first design course, but more often is left for a second design course. The reinforcement anchorage provisions of **Chapter 8** are important material for the first undergraduate design course. Students should develop a basic understanding of development length requirements for straight and hooked bars, as well as the procedure to determine bar cutoff points and reinforcement details required at those cutoff points. The serviceability requirements in **Chapter 9** for control of deflections and cracking are also important topics for the first undergraduate course. In particular, the ability to do an elastic section analysis and find moments of inertia for cracked and uncracked sections is an important skill for designers of concrete structures. **Chapter 10** serves to tie together all of the requirements for continuous floor systems introduced in Chapters 5 through 9. The examples include details for flexural and shear design, as well as full-span detailing of longitudinal and transverse reinforcement. This chapter could either be skipped for the first undergraduate course or be used as a source for a more extensive class design project. **Chapter 11** concentrates on the analysis and design of columns sections and should be included in the first undergraduate course. The portion of Chapter 11 that covers column sections subjected to biaxial bending may either be included in a first undergraduate course or saved for a graduate course. **Chapter 12** considers slenderness effects in columns, and the more detailed analysis required for this topic is commonly presented in a graduate course. If time permits, the basic information in **Chapter 15** on the design of typical concrete footings may be included in a first undergraduate course. This material may also be covered in a foundation design course taught at either the undergraduate or graduate level.

Second Design Course

Clearly, the instructor in a graduate design course has many options for topics, depending on his/her interests and the preparation of the students. **Chapter 13** is a lengthy chapter and is intended to be a significant part of a graduate course. The chapter gives extensive coverage of flexural analysis and design of two-way floor systems that builds on the analysis and design of one-way floor systems covered in Chapter 5. The direct design method and an equivalent frame method are discussed, along with more modern analysis and modeling techniques. Problems related to punching shear and the combined transfer of shear and moment at slab-to-column connections are covered in detail. The design of slab shear reinforcement, including the use of shear studs, is also presented. Finally, procedures for calculating deflections in two-way floor systems are given. Design for torsion, as given in **Chapter 7**, should be covered in conjunction with the design and analysis of two-way floor systems in Chapter 13. The design procedure for compatibility torsion at the edges of a floor system has a direct impact on the design of adjacent floor members. The presentation of the yield-line method in **Chapter 14** gives students an alternative analysis and design method for two-way slab systems. This topic could also tie in with plastic analysis methods taught in graduate level analysis courses. The analysis and design of slender columns, as presented in **Chapter 12**, should also be part of a graduate design course. The students should be prepared to apply the frame analysis and member modeling techniques required to either directly determine secondary moments or calculate the required moment-magnification factors. Also, if the topic of biaxial bending in Chapter 11 was not covered in the first design course, it could be included at this point. **Chapter 18** covers bending and shear design of structural walls that resist lateral loads due to either wind or seismic effects. A capacity-design approach is introduced for the shear design of walls that resist earthquake-induced lateral forces. **Chapter 17** covers the concept of *disturbed* regions (D-regions) and the use of the strut-and-tie models to analyze the flow of forces through D-regions and to select appropriate reinforcement details. The chapter contains detailed examples to help students learn the concepts and code requirements for strut-and-tie models. If time permits, instructors could cover the design of combined footings in **Chapter 15**, shear-friction design concepts in **Chapter 16**, and design to resist earthquake-induced forces in **Chapter 19**.

Instructor Materials

An Instructor's Solutions Manual and image PowerPoints to accompany this text are available for download to instructors only at www.pearsonhighered.com/irc.

Acknowledgment

I would like to take this opportunity to acknowledge the pioneering work done for this textbook by my former co-author, Professor James G. MacGregor. Professor MacGregor was the sole author of this textbook through its first three editions. He initiated the layout of the chapters and the presentation style of first developing an understanding of member and structural behavior, followed by derivation of specific design requirements. I used Professor MacGregor's textbook in my reinforced concrete design classes and I was very happy to join him as a co-author for the Fourth Edition. I became the primary author for the Fifth through Eighth Editions, and I owe a great deal of gratitude to Professor MacGregor for creating an outstanding textbook that I have had the privilege of modifying and enhancing. I want to give special acknowledgements to Captain Peter Amaddio, USAF and Master's Graduate from the University of Michigan for his assistance with the Solutions Manual and to Thai Dam, Ph.D. Graduate from the University of Michigan for his assistance with the column section interaction diagrams in Appendix A.

Dedication

This book is dedicated to all of my colleagues and students who have either interacted with me or taken my classes over the years. My knowledge of the behavior of reinforced concrete members and my development of various design procedures for concrete members and structures was significantly enhanced by my numerous interactions with all of you. I also wish to dedicate this book to my wife, Linda, for her support and encouragement through many long evenings and lost weekends.

The manuscript for the Fifth edition book was reviewed by Guillermo Ramirez of the University of Texas at Arlington; Devin Harris of Michigan Technological University; Sami Rizkalla of North Carolina State University; Aly Marei Said of the University of Nevada, Las Vegas; and Roberto Leon of Georgia Institute of Technology. Suggested changes for the Sixth Edition were submitted by Christopher Higgins and Thomas Schumacher of Oregon State University, Dionisio Bernal of Northeastern University, R. Paneer Selvam of the University of Arkansas, Aly Said of the University of Nevada and Chien-Chung Chen of Pennsylvania State University. Suggested changes for the Eighth Edition were submitted by Tom Panayotidi of Columbia University; Walter Gerstle of the University of New Mexico; Tongyan Pan of the University of Northwestern—St. Paul; Mustafa Mashal of Idaho State University; Patricia Rodriguez of NYU—Tandon School of Engineering; Janos Gergely of the University of North Carolina at Charlotte; and Eric E. Matsumoto of California State University, Sacramento. The book was reviewed for accuracy by Robert W. Barnes and Anton K. Schindler of Auburn University. This book was greatly improved by all of their suggestions.

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About the Author

James K. Wight received his B.S. and M.S. degrees in civil engineering from Michigan State University in 1969 and 1970, respectively, and his Ph.D. from the University of Illinois in 1973. He was a professor of structural engineering in the Civil and Environmental Engineering Department at the University of Michigan from 1973 to 2020. He taught undergraduate and graduate classes on analysis and design of reinforced concrete structures. He is well known for his work in earthquake-resistant design of concrete structures and spent a one-year sabbatical leave in Japan where he was involved in the construction and simulated earthquake testing of a full-scale reinforced concrete building. Professor Wight has been an active member of the American Concrete Institute (ACI) since 1973 and was named a Fellow of the Institute in 1984. He is a Past-President of ACI and a past Chair of the ACI Building Code Committee 318. He is also past Chair of the ACI Technical Activities Committee and Committee 352 on Joints and Connections in Concrete Structures. He has received several awards from the American Concrete Institute including the Delmar Bloem Distinguished Service Award (1991), the Joe Kelly Award (1999), the Boise Award (2002), the C.P. Siess Structural Research Award (2003 and 2009), the Alfred Lindau Award (2008), the Wason Medal (2012), and the Charles S. Whitney Medal (2015). Professor Wight has received numerous awards for his teaching and service at the University of Michigan, including the ASCE Student Chapter Teacher of the Year Award, the College of Engineering Distinguished Service Award, the College of Engineering Teaching Excellence Award, the Chi Epsilon-Great Lakes District Excellence in Teaching Award, and the Rackham Distinguished Graduate Mentoring Award. He has also received Distinguished Alumnus Awards from the Civil and Environmental Engineering Departments of the University of Illinois (2008) and Michigan State University (2009).

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1

Introduction

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1-1 REINFORCED CONCRETE STRUCTURES

Concrete and reinforced concrete are used as building construction materials in every country. In many, including the United States and Canada, reinforced concrete is a dominant structural material in engineered construction. The universal nature of reinforced concrete construction stems from the wide availability of reinforcing bars and of the constituents of concrete (gravel or crushed rock, sand, water, and cement), from the relatively simple skills required in concrete construction, and from the economy of reinforced concrete compared with other forms of construction. Plain concrete and reinforced concrete are used in buildings of all sorts (Figs. 1-1 and 1-2), underground structures, water tanks, wind turbine foundations and towers, offshore oil exploration and production structures, dams, bridges (Fig. 1-3), and even ships.

1-2 MECHANICS OF REINFORCED CONCRETE

Concrete is strong in compression, but weak in tension. As a result, cracks develop whenever loads, restrained shrinkage, or temperature changes give rise to tensile stresses in excess of the tensile strength of the concrete. In the plain concrete beam shown in Fig. 1-4b, the moments about point *O* due to applied loads are resisted by an internal tension–compression couple involving tension in the concrete. An unreinforced beam fails very suddenly and completely when the first tension crack forms. In a *reinforced concrete* beam (Fig. 1-4c), reinforcing bars are embedded in the concrete in such a way that the tension forces needed for moment equilibrium after the concrete cracks can be developed in the bars.

Alternatively, the reinforcement could be placed in a longitudinal duct near the bottom of the beam, as shown in Fig. 1-5, and stretched or *prestressed*, reacting on the concrete in the beam. This would put the reinforcement into tension and the concrete into

Fig. 1-1
Trump Tower of Chicago.
(Photograph courtesy of
Thomas Barrat/Shutterstock.)

Completed in 2009, the 92-story Trump International Hotel and Tower is an icon of the Chicago skyline. With a height of 1170 ft (1389 ft to the top of the spire), the Trump Tower is the tallest building built in North America since the completion of Sears Tower in 1974. The all reinforced concrete residential/hotel tower was designed by Skidmore, Owings & Merrill LLP (SOM). The tower's 2.6 million ft^2 of floor space is clad in stainless steel and glass, providing panoramic views of the City and Lake Michigan. The project utilized high-performance concrete mixes specified by SOM and designed by Prairie Materials Sales. The project includes self-consolidating concrete with strengths as high as 16,000 psi. The Trump Tower is not only an extremely tall structure; it is also very slender with an aspect ratio exceeding 8 to 1 (height divided by structural base dimension). Slender buildings can be susceptible to dynamic motions under wind loads. To provide the required stiffness, damping, and mass to assist in minimizing the dynamic movements, high-performance reinforced concrete was selected as the primary structural material for the tower. Lateral wind loads are resisted by a core and outrigger system. Additional torsional stiffness and structural robustness is provided by perimeter belt walls at the roof and three mechanical levels. The typical residential floor system consists of 9-in. thick flat plates with spans up to 30 ft.

compression. This compression would delay cracking of the beam. Such a member is said to be a *prestressed concrete* beam. The reinforcement in such a beam is referred to as *prestressing tendons* and must be fabricated from high-strength steel.

The construction of a reinforced concrete member involves building a form or mould in the shape of the member being constructed. The form must be strong enough to support the weight and hydrostatic pressure of the wet concrete, plus any forces applied to it by workers, concrete casting equipment, wind, and so on. The reinforcement is placed in the form and held in place during the concreting operation. After the concrete has reached sufficient strength, the forms can be removed.

Fig. 1-2
Overview of 432 Park Avenue after completion of construction. (Photograph courtesy of Gabor Kovacs Photography/Shutterstock.)

When completed in 2017, 432 Park (432 Park Avenue in Midtown Manhattan) was the tallest residential tower (approximately 1400 feet) in the western hemisphere. More than 70,000 cubic yards of concrete and 12,500 tons of reinforcing steel was used for construction of the superstructure. Higher strength concrete (up to 14,000 psi) was used near the base of the structure for increased stiffness and smaller member sizes. Columns and shear walls were constructed with high-strength reinforcing steel (up to Grade 100) that was spliced with mechanical connectors. Exposed structural members were cast with white Portland cement, and a sustainable concrete mixture that replaced up to 70% of Portland cement with pozzolanic materials was used for the interior shear wall core.

1-3 REINFORCED CONCRETE MEMBERS

Reinforced concrete structures consist of a monolithic series of "members" that interact to support loads placed on the structure. The second floor of the building in Fig. 1-6 is built of concrete joist–slab construction. Here, a series of parallel ribs or *joists* support the load from the top slab. The reactions supporting the joists apply loads to the beams, which in turn are supported by columns. In such a floor, the top slab has two functions: (1) it transfers load laterally to the joists, and (2) it serves as the top flange of the joists, which act as T-shaped beams that transmit load to the beams running at right angles to the joists. The first floor of the building in Fig. 1-6 has a slab-and-beam design in which the slab spans between beams, which in turn apply loads to the columns. The column loads are applied to *spread footings*, which distribute the load over an area of soil sufficient to prevent overloading of the soil. Some soil conditions require the use of pile foundations or other deep foundations. At the perimeter of the building, the floor loads are supported either directly on the walls, as shown in Fig. 1-6, or on exterior columns, as shown in Fig. 1-7. The walls, in turn, are supported by a basement wall and wall footings.

Fig. 1-3
St. Anthony Falls Bridge.
(Photograph courtesy of Photo Image/Shutterstock.)

The new I-35W Bridge (St. Anthony Falls Bridge) in Minneapolis, Minnesota, features a 504 ft main span over the Mississippi River. The concrete piers and superstructure were shaped to echo the arched bridges and natural features in the vicinity. The bridge was designed by FIGG Bridge Engineers, Inc. and constructed by Flatiron-Manson Joint Venture in less than 14 months after the tragic collapse of the former bridge at this site. Segmentally constructed post-tensioned box girders with a specified concrete strength of 6500 psi were used for the bridge superstructure. The tapered piers were cast-in-place and used a specified concrete strength of 4000 psi. Also, a new self-cleaning pollution-eating concrete was used to construct two 30-ft gateway sculptures located at each end of the bridge. A total of approximately 50,000 cubic yards of concrete and 7000 tons of reinforcing bars and post-tensioning steel were used in the project.

The first- and second-floor slabs in Fig. 1-6 are assumed to carry the loads in a north–south direction (see direction arrow) to the joists or beams, which carry the loads in an east–west direction to other beams, girders, columns, or walls. This is referred to as *one-way slab* action and is analogous to a wooden floor in a house, in which the floor decking transmits loads to perpendicular floor joists, which carry the loads to supporting beams, and so on.

The ability to form and construct concrete slabs makes possible the slab or plate type of structure shown in Fig. 1-7. Here, the loads applied to the roof and the floor are transmitted in two directions to the columns by plate action. Such slabs are referred to as *two-way slabs*.

The first floor in Fig. 1-7 is a *flat slab* with thickened areas called *drop panels* at the columns. In addition, the tops of the columns are enlarged in the form of *capitals* or *brackets*. The thickening provides extra depth for moment and shear resistance adjacent to the columns. It also tends to reduce slab deflections.

The roof of the building shown in Fig. 1-7 is of uniform thickness throughout, without drop panels or column capitals. Such a floor is a special type of *flat slab* referred to as a *flat plate*. Flat-plate floors are widely used in apartments because the underside of the slab is flat and hence, can be used as the ceiling of the room below. Of equal importance, the forming for a flat plate is generally cheaper than that for flat slabs with drop panels or for one-way slab-and-beam floors.

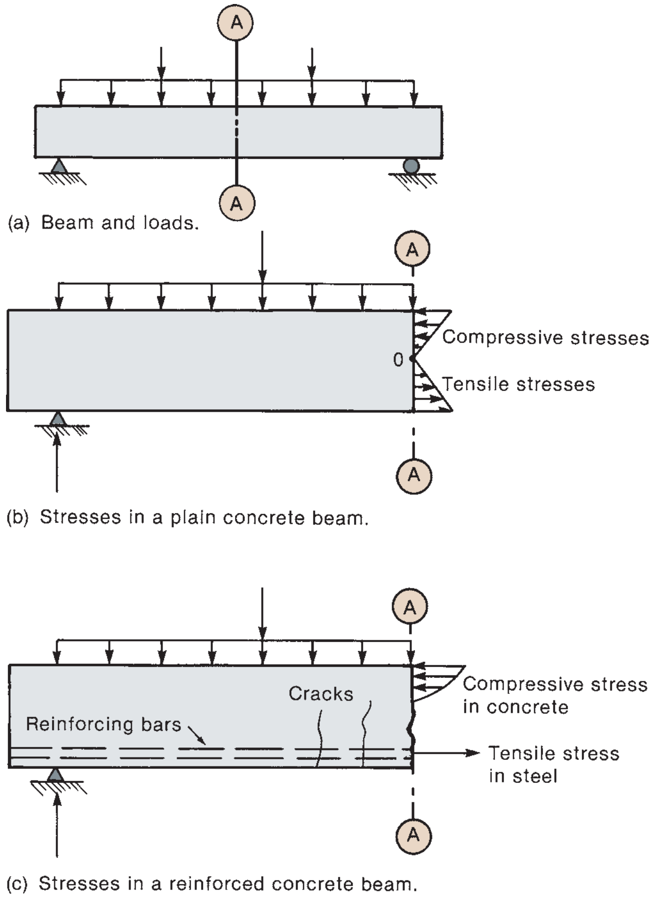

Fig. 1-4
Plain and reinforced concrete beams.

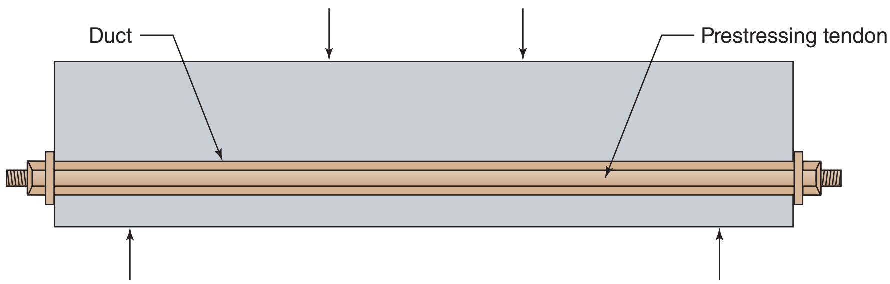

Fig. 1-5
Prestressed concrete beam.

1-4 FACTORS AFFECTING CHOICE OF REINFORCED CONCRETE FOR A STRUCTURE

The choice of whether a structure should be built of reinforced concrete, steel, masonry, or timber depends on the availability of materials and on a number of value decisions.

1. Economy. Frequently, the foremost consideration is the overall cost of the structure. This is, of course, a function of the costs of the materials and of the labor and time necessary to erect the structure. Concrete floor systems tend to be thinner than structural steel systems because the girders and beams or joists all fit within the same depth, as

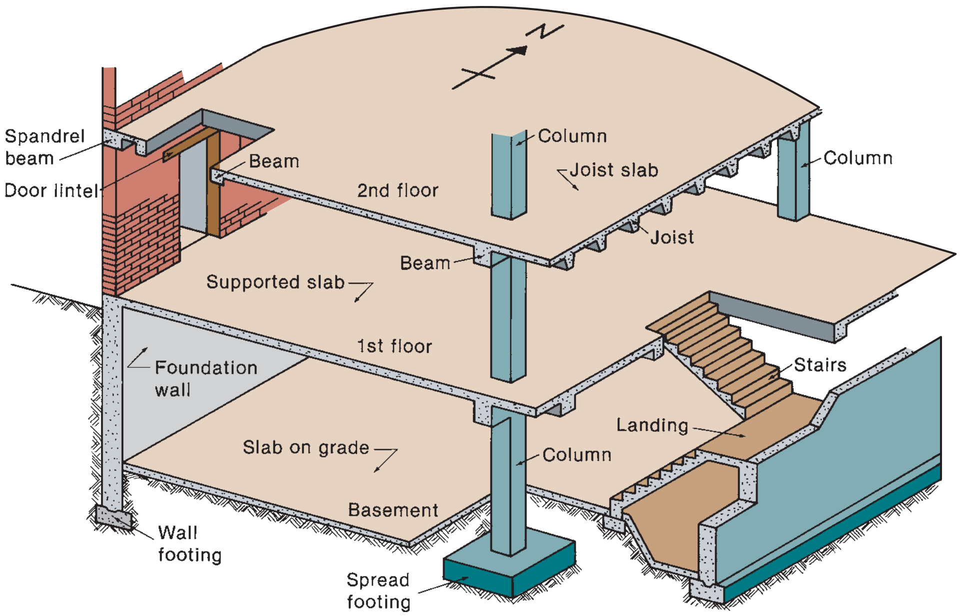

Fig. 1-6
Reinforced concrete
building elements.

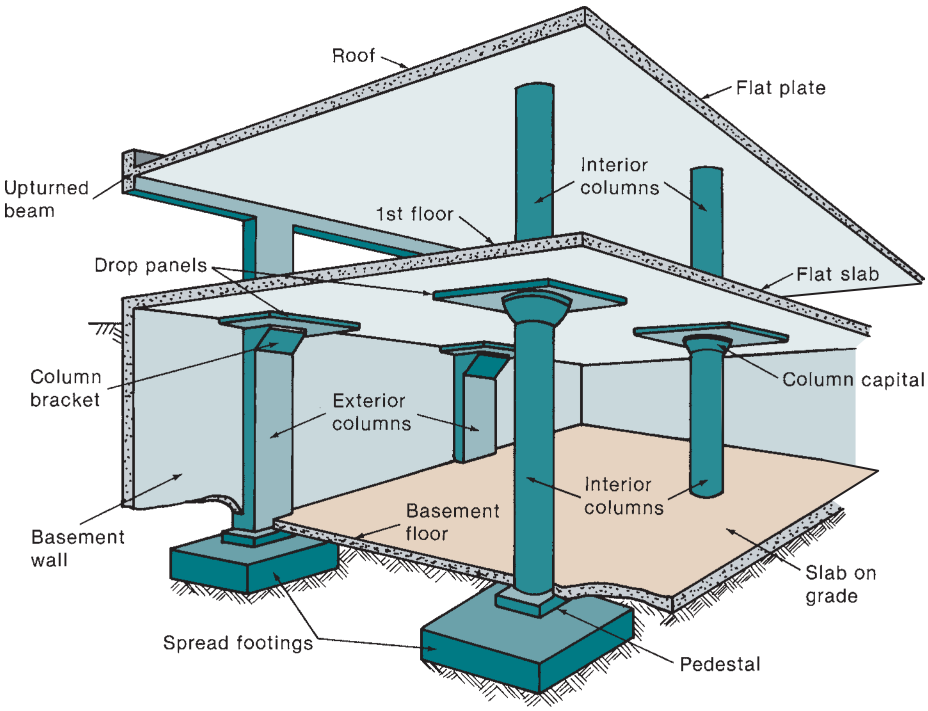

Fig. 1-7
Reinforced concrete building
elements.

shown in the second floor in Fig. 1-6, or the floors are flat plates or flat slabs, as shown in Fig. 1-7 [1-1]. This produces an overall reduction in the height of a building compared to a steel building, which leads to (a) lower wind loads because there is less area exposed to wind and (b) savings in cladding and mechanical and electrical risers.

Frequently, however, the overall cost is affected as much or more by the overall construction time, because the contractor and the owner must allocate money to carry out the construction and will not receive a return on their investment until the building is ready for occupancy. As a result, financial savings due to rapid construction may more than offset increased material and forming costs. The materials for reinforced concrete structures are widely available and can be produced as they are needed in the construction, whereas structural steel must be ordered and partially paid for in advance to schedule the job in a steel-fabricating yard.

Any measures the designer can take to standardize the design and forming will generally pay off in reduced overall costs. For example, column sizes may be kept the same for several floors to save money in form costs, while changing the concrete strength or the percentage of reinforcement allows for changes in column loads.

2. Suitability of material for architectural and structural function. A reinforced concrete system frequently allows the designer to combine the architectural and structural functions. Concrete has the advantage that it is placed in a plastic condition and is given the desired shape and texture by means of the forms and the finishing techniques. This allows such elements as flat plates or other types of slabs to serve as load-bearing elements while providing the finished floor and ceiling surfaces. Similarly, reinforced concrete walls can provide architecturally attractive surfaces in addition to having the ability to resist gravity, wind, or seismic loads. Finally, the choice of size or shape is governed by the designer and not by the availability of standard manufactured members.

3. Fire resistance. The structure in a building must withstand the effects of a fire and remain standing while the building is being evacuated and the fire extinguished. A concrete building inherently has a 1- to 3-hour fire rating without special fireproofing or other details. Structural steel or timber buildings must be fireproofed to attain similar fire ratings.

4. Rigidity. The occupants of a building may be disturbed if their building oscillates in the wind or if the floors vibrate as people walk by. Due to the greater stiffness and mass of a concrete structure, such vibrations are seldom a problem.

5. Low maintenance. Concrete members inherently require less maintenance than do structural steel or timber members. This is particularly true if dense, air-entrained concrete has been used for surfaces exposed to the atmosphere and if care has been taken in the design to provide adequate drainage from the structure.

6. Availability of materials. Sand, gravel or crushed rock, water, cement, and concrete mixing facilities are very widely available, and reinforcing steel can be transported to most construction sites more easily than can structural steel. As a result, reinforced concrete is frequently the preferred construction material in remote areas.

On the other hand, there are a number of factors that may cause one to select a material other than reinforced concrete. These include:

1. Low tensile strength. As stated earlier, the tensile strength of concrete is much lower than its compressive strength (about $\frac{1}{10}$); hence, concrete is subject to cracking when subjected to tensile stresses. In structural uses, the cracking is restrained by using reinforcement, as shown in Fig. 1-4c, to carry tensile forces and limit crack widths to within acceptable values. Unless care is taken in design and construction, however, these cracks may be unsightly or may allow penetration of water and other potentially harmful contaminants.

2. Forms and shoring. The construction of a cast-in-place structure involves three steps not encountered in the construction of steel or timber structures. These are: (a) the construction of the forms, (b) the removal of these forms, and (c) the propping or shoring of the new concrete to support its weight until its strength is adequate. Each of these steps involves labor and/or materials that are not necessary with other forms of construction.

3. Relatively low strength per unit of weight or volume. The compressive strength of concrete is roughly 10 percent that of steel, while its unit density is roughly 30 percent that of steel. As a result, a concrete structure requires a larger volume and a greater weight of material than does a comparable steel structure.

4. Time-dependent volume changes. Both concrete and steel undergo approximately the same amount of thermal expansion and contraction. Because there is less mass of steel to be heated or cooled, and because steel is a better conductor than concrete, a steel structure is generally affected by temperature changes to a greater extent than is a concrete structure. On the other hand, concrete undergoes drying shrinkage, which, if restrained, may cause cracking. Furthermore, deflections in a concrete floor will tend to increase with time, possibly doubling, due to creep of the concrete under sustained compression stress.

1-5 HISTORICAL DEVELOPMENT OF CONCRETE AND REINFORCED CONCRETE AS STRUCTURAL MATERIALS

Cement and Concrete

Lime mortar was first used in structures in the Minoan civilization in Crete about 2000 B.C. and is still used in some areas. This type of mortar had the disadvantage of gradually dissolving when immersed in water and hence could not be used for exposed or underwater structural members. About the third century B.C., the Romans discovered a fine sandy volcanic ash that, when mixed with lime mortar, gave a much stronger mortar, which could be used under water.

One of the most remarkable concrete structures built by the Romans was the dome of the Pantheon in Rome, completed in A.D. 126. This dome has a span of 144 ft, a span not exceeded until the nineteenth century. The lowest part of the dome was concrete with aggregate consisting of broken bricks. As the builders approached the top of the dome they used lighter and lighter aggregates, using pumice at the top to reduce the dead-load moments. Although the outside of the dome was, and still is, covered with decorations, the marks of the forms are still visible on the inside [1-2], [1-3].

While designing the Eddystone Lighthouse off the south coast of England just before A.D. 1800, the English engineer John Smeaton discovered that a mixture of burned limestone and clay could be used to make a cement that would set under water and be water resistant. Owing to the exposed nature of this lighthouse, however, Smeaton reverted to the tried-and-true Roman cement and mortised stonework.

In the ensuing years a number of people used Smeaton's material, but the difficulty of finding limestone and clay in the same quarry greatly restricted its use. In 1824, Joseph Aspdin mixed ground limestone and clay from different quarries and heated them in a kiln to make cement. Aspdin named his product Portland cement because concrete made from it resembled Portland stone, a high-grade limestone from the Isle of Portland in the south of England. This cement was used by Brunel in 1828 for the mortar in the masonry liner of a tunnel under the Thames River and in 1835 for mass concrete piers for a bridge. Occasionally in the production of cement, the mixture would be overheated, forming a hard clinker, which was considered to be spoiled and was discarded. In 1845, I. C. Johnson found that the best cement resulted from grinding this clinker. This is the material now known as Portland cement. Portland cement was produced in Pennsylvania in 1871 by D. O. Saylor and about the same time in Indiana by T. Millen of South Bend, but it was not until the early 1880s that significant amounts were produced in the United States.

Reinforced Concrete

W. B. Wilkinson of Newcastle-upon-Tyne obtained a patent in 1854 for a reinforced concrete floor system that used hollow plaster domes as forms. The ribs between the forms were filled with concrete and were reinforced with discarded steel mine-hoist ropes in the center of the ribs. In France, Lambot built a rowboat of concrete reinforced with wire in 1848 and patented it in 1855. His patent included drawings of a reinforced concrete beam and a column reinforced with four round iron bars. In 1861, another Frenchman, Coignet, published a book illustrating uses of reinforced concrete.

The American lawyer and engineer Thaddeus Hyatt experimented with reinforced concrete beams in the 1850s. His beams had longitudinal bars in the tension zone and vertical stirrups for shear. Unfortunately, Hyatt's work was not known until he privately published a book describing his tests and building system in 1877.

Perhaps the greatest incentive to the early development of the scientific knowledge of reinforced concrete came from the work of Joseph Monier, owner of a French nursery garden. Monier began experimenting in about 1850 with concrete tubs reinforced with iron for planting trees. He patented his idea in 1867. This patent was rapidly followed by patents for reinforced pipes and tanks (1868), flat plates (1869), bridges (1873), and stairs (1875). In 1880 and 1881, Monier received German patents for many of the same applications. These were licensed to the construction firm Wayss and Freitag, which commissioned Professors Mörsch and Bach of the University of Stuttgart to test the strength of reinforced concrete and commissioned Mr. Koenen, chief building inspector for Prussia, to develop a method for computing the strength of reinforced concrete. Koenen's book, published in 1886, presented an analysis that assumed the neutral axis was at the midheight of the member.

The first reinforced concrete building in the United States was a house built on Long Island in 1875 by W. E. Ward, a mechanical engineer. E. L. Ransome of California experimented with reinforced concrete in the 1870s and patented a twisted steel reinforcing bar in 1884. In the same year, Ransome independently developed his own set of design procedures. In 1888, he constructed a building having cast-iron columns and a reinforced concrete floor system consisting of beams and a slab made from flat metal arches covered with concrete. In 1890, Ransome built the Leland Stanford, Jr. Museum in San Francisco. This two-story building used discarded cable-car rope as beam reinforcement. In 1903 in Pennsylvania, he built the first building in the United States completely framed with reinforced concrete.

In the period from 1875 to 1900, the science of reinforced concrete developed through a series of patents. An English textbook published in 1904 listed 43 patented systems, 15 in France, 14 in Germany or Austria–Hungary, 8 in the United States, 3 in the United Kingdom, and 3 elsewhere. Most of these differed in the shape of the bars and the manner in which the bars were bent.

From 1890 to 1920, practicing engineers gradually gained a knowledge of the mechanics of reinforced concrete, as books, technical articles, and codes presented the theories. In an 1894 paper to the French Society of Civil Engineers, Coignet (son of the earlier Coignet) and de Tedeskko extended Koenen's theories to develop the working-stress design method for flexure, which was used universally from 1900 to 1950. During the past seven decades, extensive research has been carried out on various aspects of reinforced concrete behavior, resulting in the current design procedures.

Prestressed concrete was pioneered by E. Freyssinet, who in 1928 concluded that it was necessary to use high-strength steel wire for prestressing because the creep of concrete dissipated most of the prestress force if normal reinforcing bars were used to develop the prestressing force. Freyssinet developed anchorages for the tendons and designed and built a number of pioneering bridges and structures.

Design Specifications for Reinforced Concrete

The first set of building regulations for reinforced concrete were drafted under the leadership of Professor Mörsch of the University of Stuttgart and were issued in Prussia in 1904. Design regulations were issued in Britain, France, Austria, and Switzerland between 1907 and 1909.

The American Railway Engineering Association appointed a Committee on Masonry in 1890. In 1903 this committee presented specifications for portland cement concrete. Between 1908 and 1910, a series of committee reports led to the *Standard Building Regulations for the Use of Reinforced Concrete*, published in 1910 [1-4] by the National Association of Cement Users, which subsequently became the American Concrete Institute.

A Joint Committee on Concrete and Reinforced Concrete was established in 1904 by the American Society of Civil Engineers, the American Society for Testing and Materials, the American Railway Engineering Association, and the Association of American Portland Cement Manufacturers. This group was later joined by the American Concrete Institute. Between 1904 and 1910, the Joint Committee carried out research. A preliminary report issued in 1913 [1-5] lists the more important papers and books on reinforced concrete published between 1898 and 1911. The final report of this committee was published in 1916 [1-6]. The history of reinforced concrete building codes in the United States was reviewed in 1954 by Kerekes and Reid [1-7].

1-6 BUILDING CODES AND THE ACI CODE

The design and construction of buildings is regulated by municipal bylaws called *building codes*. These exist to protect the public's health and safety. Each city and town is free to write or adopt its own building code, and in that city or town, only that particular code has legal status. Because of the complexity of writing building codes, cities in the United States generally base their building codes on model codes. Prior to the year 2000, there were three model codes: the *Uniform Building Code* [1-8], the *Standard Building Code* [1-9], and the *Basic Building Code* [1-10]. These codes covered such topics as use and occupancy requirements, fire requirements, heating and ventilating requirements, and structural design. In 2000, these three codes were replaced by the *International Building Code* (*IBC*) [1-11], which is normally updated every three to five years.

The definitive design specification for reinforced concrete buildings in North America is the *Building Code Requirements for Structural Concrete (ACI 318-19)* and *Commentary (ACI 318R-19)* [1-12]. The code and the commentary are bound together in one volume.

This code, generally referred to as the *ACI Code*, has been incorporated by reference in the *International Building Code* and serves as the basis for comparable codes in Canada, New Zealand, Australia, most of Latin America, and some countries in Asia and the Middle East. The ACI Code has legal status only if adopted in a local building code.

In prior years, the ACI Code had a major revision every three years. Current plans are to publish major revisions on a five- or six-year cycle. This book refers extensively to the 2019 ACI Code. It is recommended that the reader have a copy available.

The term *structural concrete* is used to refer to the entire range of concrete structures: from *plain concrete* with limited reinforcement; through *ordinary reinforced concrete*, reinforced with normal reinforcing bars; through *partially prestressed concrete*, generally containing both reinforcing bars and prestressing tendons; to *fully prestressed concrete*, with enough prestress to prevent cracking in everyday service. In 1995, the title of the ACI Code was changed from *Building Code Requirements for Reinforced Concrete* to *Building Code Requirements for Structural Concrete* to emphasize that the code deals with the entire spectrum of structural concrete.

The rules for the design of concrete highway bridges are specified in the *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington, D.C. [1-13].

Another document that will be used in Chapters 2 and 19 is the ASCE standard *ASCE/SEI 7-16*, entitled *Minimum Design Loads for Buildings and Other Structures* [1-14], published in 2017.

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2

The Design Process

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2-1 OBJECTIVES OF DESIGN

A structural engineer is a member of a team that works together to design a building, bridge, or other structure. In the case of a building, an architect generally provides the overall layout, and mechanical, electrical, and structural engineers design individual systems within the building.

The structure should satisfy four major criteria:

1. **Appropriateness.** The arrangement of spaces, spans, ceiling heights, access, and traffic flow must complement the intended use. The structure should fit its environment and be aesthetically pleasing.

2. **Economy.** The overall cost of the structure should not exceed the client's budget. Normally, coordination between the designs of individual systems within the structure will result in cost savings.

3. **Structural adequacy.** Structural adequacy involves two major aspects.

   (a) A structure must be strong enough to support all anticipated loadings safely.

   (b) A structure must not deflect, tilt, vibrate, or crack in a manner that impairs its usefulness.

4. **Maintainability.** A structure should be designed so as to require a minimum amount of simple maintenance procedures.

2-2 THE DESIGN PROCESS

The design process is a sequential and iterative decision-making process. The three major phases are the following:

1. **Definition of the client's needs and priorities.** All buildings or other structures are built to fulfill a need. It is important that the owner or user be involved in determining the attributes of the proposed building. These include functional requirements, aesthetic requirements, and budgetary requirements. The latter include initial cost, premium for rapid construction to allow early occupancy, maintenance, and other life-cycle costs.

2. Development of project concept. Based on the client's needs and priorities, a number of possible layouts are developed. Preliminary cost estimates are made and the final choice of the system to be used is based on how well the overall design satisfies the client's needs within the budget available. Generally, systems that are conceptually simple and have standardized geometries and details that allow construction to proceed as a series of identical cycles are the most cost effective.

During this stage, the overall structural concept is selected. From approximate analyses of the moments, shears, and axial forces, preliminary member sizes are selected for each potential scheme. Once this is done, it is possible to estimate costs and select the most desirable structural system.

The overall thrust in this stage of the structural design is to satisfy the design criteria dealing with appropriateness, economy, and maintainability.

3. Design of individual systems. Once the overall layout and general structural concept have been selected, the structural system can be designed. Structural design involves three main steps. Based on the preliminary design selected in phase 2, a *structural analysis* is carried out to determine the moments, shears, torques, and axial forces in the structure. The individual members are then *proportioned* to resist these load effects. The proportioning, sometimes referred to as *member design*, must also consider overall aesthetics, the constructability of the design, coordination with mechanical and electrical systems, and the sustainability of the final structure. The final stage in the design process is to prepare construction drawings and specifications.

2-3 LIMIT STATES AND THE DESIGN OF REINFORCED CONCRETE

Limit States

When a structure or structural element becomes unfit for its intended use, it is said to have reached a *limit state*. The limit states for reinforced concrete structures can be divided into three basic groups:

1. Strength limit states. These involve a structural collapse of part or all of the structure. Such a limit state should have a very low probability of occurrence, because it may lead to loss of life and major financial losses. The major strength limit states are as follows:

(a) Loss of equilibrium of a part or all of the structure as a rigid body. Such a failure would generally involve tipping or sliding of the entire structure and would occur if the reactions necessary for equilibrium could not be developed.

(b) Failure of critical parts of the structure, leading to partial or complete collapse. The majority of this book deals with this limit state. Chapters 4 and 5 consider flexural failures; Chapter 6, shear failures; and so on.

(c) Progressive collapse. In some structures, an overload on one member may cause that member to fail. The load acting on it is transferred to adjacent members which, in turn, may be overloaded and fail, causing them to shed their load to adjacent members, causing them to fail one after another, until a major part of the structure has collapsed. This is called a *progressive collapse* [2-1], [2-2]. Progressive collapse is prevented, or at least is limited, by one or more of the following:

(i) Controlling accidental events by taking measures such as protection against vehicle collisions or explosions.

(ii) Providing local resistance by designing key members to resist accidental events.

(iii) Providing minimum horizontal and vertical ties to transfer forces.

(iv) Providing alternative lines of support to anchor the tie forces.

(v) Limiting the spread of damage by subdividing the building with planes of weakness, sometimes referred to as *structural fuses*.

A structure is said to have *general structural integrity* if it is resistant to progressive collapse. For example, a terrorist bomb or a vehicle collision may accidentally remove a column that supports an interior support of a two-span continuous beam. If properly detailed, the structural system may change from two spans to one long span. This would entail large deflections and a change in the load path from beam action to *catenary* or tension membrane action. ACI Code Section 9.7.7 requires continuous tensile reinforcement in beams around the perimeter of a building at each floor to reduce the risk of progressive collapse. This continuous reinforcement anchors the catenary forces and limits the spread of damage. Because such failures are most apt to occur during construction, the designer should be aware of the applicable construction loads and procedures.

(d) Formation of a plastic mechanism. A mechanism is formed when the reinforcement yields to form plastic hinges at enough sections to make the structure unstable.

(e) Instability due to deformations of the structure. This type of failure involves buckling and is discussed more fully in Chapter 12.

(f) Fatigue. Fracture of members due to repeated stress cycles of service loads may cause collapse. Fatigue is discussed in Sections 3-14 and 9-8.

2. Serviceability limit states. These involve disruption of the functional use of the structure, but not collapse per se. Because there is less danger of loss of life, a higher probability of occurrence can generally be tolerated than in the case of a strength limit state. Design for serviceability is discussed in Chapter 9. The major serviceability limit states include the following:

(a) Excessive deflections for normal service. Excessive deflections may cause machinery to malfunction, may be visually unacceptable, and may lead to damage to nonstructural elements or to changes in the distribution of forces. In the case of very flexible roofs, deflections due to the weight of water on the roof may lead to increased depth of water, increased deflections, and so on, until the strength of the roof is exceeded. This is a *ponding failure* and in essence is a collapse brought about by failure to satisfy a serviceability limit state.

(b) Excessive crack widths. Although reinforced concrete must crack before the reinforcement can function effectively, it is possible to detail the reinforcement to minimize crack widths. Excessive crack widths may be unsightly and may allow leakage through the cracks, corrosion of the reinforcement, and gradual deterioration of the concrete.

(c) Undesirable vibrations. Vertical vibrations of floors or bridges and lateral and torsional vibrations of tall buildings may disturb the users. Vibration effects have rarely been a problem in reinforced concrete buildings.

3. Special limit states. This class of limit states involves damage or failure due to abnormal conditions or abnormal loadings and includes:

(a) damage or collapse in extreme earthquakes,

(b) structural effects of fire, explosions, or vehicular collisions,

(c) structural effects of reinforcement corrosion or concrete deterioration, and

(d) long-term physical or chemical instability (normally not a problem with concrete structures).

Limit-States Design

Limit-states design is a process that involves

1. the identification of all potential modes of failure (i.e., identification of the significant limit states),

2. the determination of acceptable levels of safety against occurrence of each limit state, and

3. structural design for the significant limit states.

For normal structures, step 2 is carried out by the building-code authorities, who specify the load combinations and the load factors to be used. For unusual structures, the engineer may need to check whether the normal levels of safety are adequate.

For buildings, a limit-states design starts by selecting the concrete strength, cement content, cement type, supplementary cementitious materials, water–cementitious materials ratio, air content, and cover to the reinforcement to satisfy the durability requirements of ACI Code Chapter 19. Next, the minimum member sizes and minimum covers are chosen to satisfy the fire-protection requirements of the local building code. Design is then carried out, starting by proportioning for the ultimate limit states followed by a check of whether the structure will exceed any of the serviceability limit states. This sequence is followed because the major function of structural members in buildings is to resist loads without endangering the occupants. For a water tank, however, the limit state of excessive crack widths is of equal importance to any of the strength limit states if the structure is to remain watertight [2-3]. In such a structure, the design for the limit state of crack width might be considered before the strength limit states are checked. In the design of support beams for an elevated monorail, the smoothness of the ride is extremely important, and the limit state of deflection may govern the design.

Basic Design Relationship

Figure 2-1a shows a beam that supports its own dead weight, w, plus some applied loads, P_1, P_2, and P_3. These cause bending moments, distributed as shown in Fig. 2-1b. The bending moments are obtained directly from the loads by using the laws of statics, and for a known span and combination of loads w, P_1, P_2, and P_3, the moment diagram is independent of the composition or shape of the beam. The bending moment is referred to as a *load effect*. Other load effects include shear force, axial force, torque, deflection, and vibration.

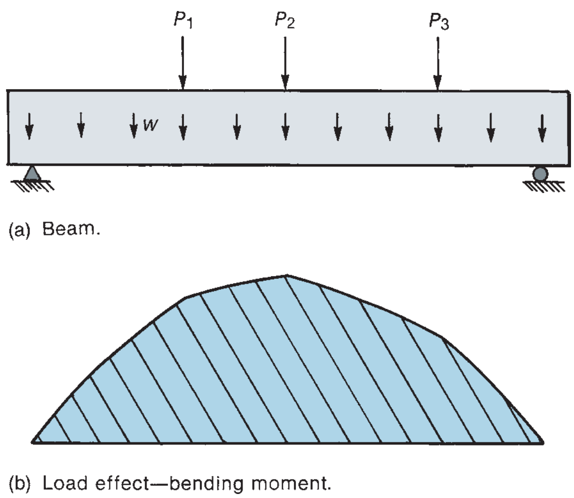


Fig. 2-1
Beam with loads and a load effect.

Figure 2-2a shows flexural stresses acting on a beam cross section. The compressive and tensile stress blocks in Fig. 2-2a can be replaced by forces C and T that are separated by a distance jd, as shown in Fig. 2-2b. The resulting couple is called an *internal resisting moment*. The internal resisting moment when the cross section fails is referred to as the *moment strength* or *moment resistance*. The word *strength* also can be used to describe shear strength and axial load strength.

The beam shown in Fig. 2-2 will support the loads safely if, at every section, the resistance (strength) of the member exceeds the effects of the loads:

$$\text{resistances} \geq \text{load effects} \tag{2-1}$$

To allow for the possibility that the resistances will be less than computed or the load effects larger than computed, *strength-reduction factors*, ϕ, less than 1, and *load factors,* α, greater than 1, are introduced:

$$\phi R_n \geq \alpha_1 S_1 + \alpha_2 S_2 + \cdots \tag{2-2a}$$

Here, R_n stands for nominal resistance (strength) and S stands for load effects based on the specified loads. Written in terms of moments, (2-2a) becomes

$$\phi_M M_n \geq \alpha_D M_D + \alpha_L M_L + \cdots \tag{2-2b}$$

where M_n is the *nominal moment strength*. The word *nominal* implies that this strength is a computed value based on the specified concrete and steel strengths and the dimensions shown on the drawings. M_D and M_L are the bending moments (load effects) due to the specified dead load and specified live load, respectively; ϕ_M is a strength-reduction factor for moment; and α_D and α_L are load factors for dead and live load, respectively.

Similar equations can be written for shear, V, and axial force, P:

$$\phi_V V_n \geq \alpha_D V_D + \alpha_L V_L + \cdots \tag{2-2c}$$

$$\phi_P P_n \geq \alpha_D P_D + \alpha_L P_L + \cdots \tag{2-2d}$$

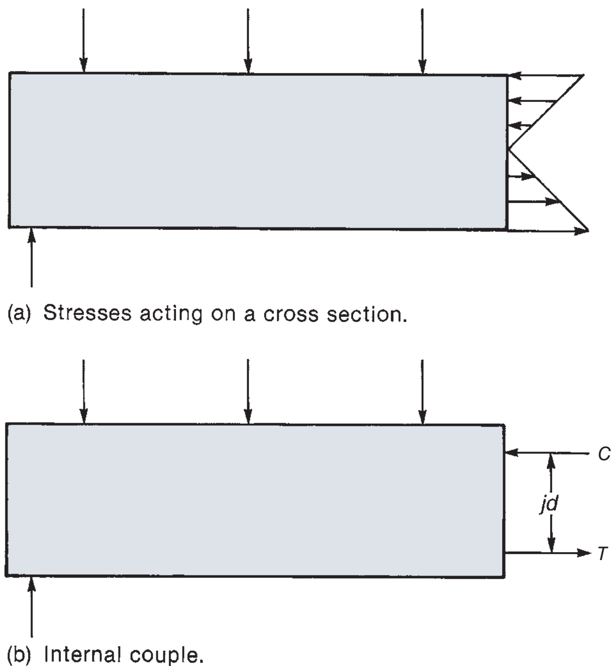


Fig. 2-2
Internal resisting moment.

Equation (2-1) is the basic limit-states design equation. Equations (2-2a) to (2-2d) are special forms of this basic equation. Throughout the ACI Code, the symbol U is used to refer to the load combination $(\alpha_D D + \alpha_L L + \cdots)$. This combination is referred to as the *factored loads*. The symbols M_u, V_u, T_u, and so on, refer to *factored-load effects* calculated from the factored loads.

2-4 STRUCTURAL SAFETY

There are three main reasons why safety factors, such as load and resistance factors, are necessary in structural design:

1. Variability in strength. The actual strengths (resistances) of beams, columns, or other structural members will almost always differ from the values calculated by the designer. The main reasons for this are as follows [2-4]:

(a) variability of the strengths of concrete and reinforcement,

(b) differences between the as-built dimensions and those shown on the structural drawings, and

(c) effects of simplifying assumptions made in deriving the equations for member strength.

A histogram of the ratio of beam moment capacities observed in tests, M_{test}, to the nominal strengths computed by the designer, M_n, is plotted in Fig. 2-3. Although the mean strength is roughly 1.05 times the nominal strength in this sample, there is a definite chance that some beam cross sections will have a lower capacity than computed. The variability shown here is due largely to the simplifying assumptions made in computing the nominal moment strength, M_n.

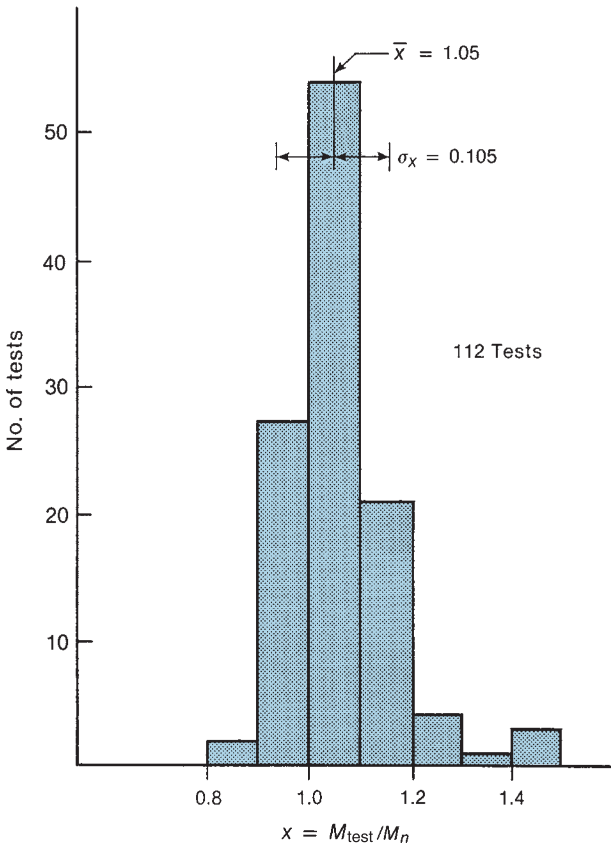


Fig. 2-3
Comparison of measured and computed failure moments, based on data for reinforced concrete beams with $f'_c > 2000$ psi [2-5].

2. Variability in loadings. All loadings are variable, especially live loads and environmental loads due to snow, wind, or earthquakes. Figure 2-4a compares the sustained component of live loads measured in a series of 151-ft^2 areas in offices. Although the average sustained live load was 13 psf in this sample, 1 percent of the measured loads exceeded 44 psf. For this type of occupancy and area, building codes specify live loads of 50 psf. For larger areas, the mean sustained live load remains close to 13 psf, but the variability decreases, as shown in Fig. 2-4b. A transient live load representing unusual loadings due to parties, temporary storage, and so on, must be added to get the total live load. As a result, the maximum live load on a given office will generally exceed the 13 to 44 psf discussed here.

In addition to actual variations in the loads themselves, the assumptions and approximations made in carrying out structural analyses lead to differences between the actual forces and moments and those computed by the designer [2-4]. Due to the variabilities of strengths and load effects, there is a definite chance that a weaker-than-average structure will be subjected to a higher-than-average load, and in this extreme case, failure may occur. The load factors and resistance (strength) factors in Eqs. (2-2a) through (2-2d) are selected to reduce the probability of failure to a very small level.

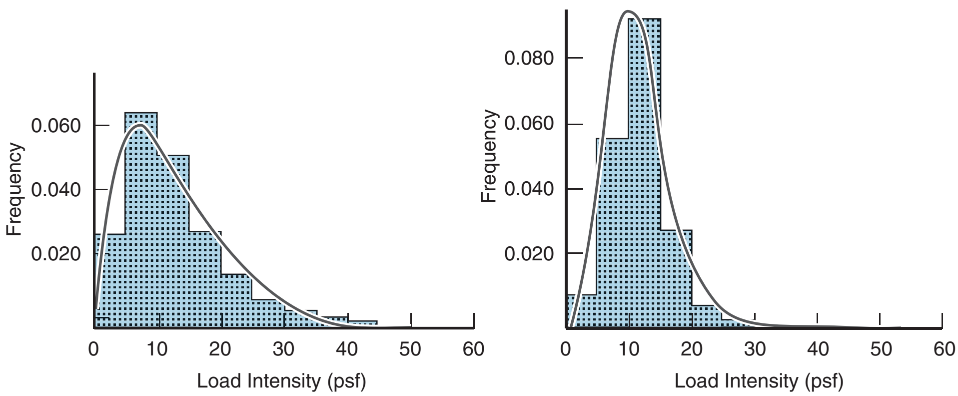
The consequences of failure are a third factor that must be considered in establishing the level of safety required in a particular structure.

3. Consequences of failure. A number of subjective factors must be considered in determining an acceptable level of safety for a particular class of structure. These include:

(a) The potential loss of life—it may be desirable to have a higher factor of safety for an auditorium than for a storage building.

(b) The cost to society in lost time, lost revenue, or indirect loss of life or property due to a failure—for example, the failure of a bridge may result in intangible costs due to traffic congestion that could approach the replacement cost.

(c) The type of failure, warning of failure, and existence of alternative load paths—if the failure of a member is preceded by excessive deflections, as in the case of a ductile flexural failure of a reinforced concrete beam, the persons endangered by the impending collapse will be warned and will have a chance to leave the building prior to failure. This may not be possible if a member fails suddenly without warning, as may be the case for a compression failure in a tied column. Thus, the required level of safety may not need to be as high for a beam as for a column. In some structures, the yielding or failure of one member causes a redistribution of load to adjacent members. In other structures, the failure of one



(a) Area = 151 ft^2.

(b) Area = 2069 ft^2.

Fig. 2-4
Frequency distribution of sustained component of live loads in offices. (From [2-6].)

member causes complete collapse. If no redistribution is possible, a higher level of safety is required.

(d) The direct cost of clearing the debris and replacing the structure and its contents.

2-5 PROBABILISTIC CALCULATION OF SAFETY FACTORS

The distribution of a population of resistances, R, of a group of similar structures is plotted on the horizontal axis in Fig. 2-5. This is compared to the distribution of the maximum load effects, S, expected to occur on those structures during their lifetimes, plotted on the vertical axis in the same figure. For consistency, both the resistances and the load effects can be expressed in terms of a quantity such as bending moment. The 45° line in this figure corresponds to a load effect equal to the resistance. Combinations of S and R falling above this line correspond to $S > R$ and, hence, failure. Thus, load effect S_1 acting on a structure having strength R_1 would cause failure, whereas load effect S_2 acting on a structure having resistance R_2 represents a safe combination.

For a given distribution of load effects, the probability of failure can be reduced by increasing the resistances. This would correspond to shifting the distribution of resistances to the right in Fig. 2-5. The probability of failure also could be reduced by reducing the dispersion of the resistances.

The term $Y = R - S$ is called the *safety margin*. By definition, failure will occur if Y is negative, represented by the shaded area in Fig. 2-6. The *probability of failure*, P_f, is the chance that a particular combination of R and S will give a negative value of Y. This probability is equal to the ratio of the shaded area to the total area under the curve in Fig. 2-6. This can be expressed as

$$P_f = \text{probability that } [Y < 0] \tag{2-3}$$

The function Y has mean value $\overline{Y}$ and standard deviation σ_Y. From Fig. 2-6, it can be seen that $\overline{Y} = 0 + \beta\sigma_Y$, where $\beta = \overline{Y}/\sigma_Y$. If the distribution is shifted to the right by increasing the resistance, thereby making $\overline{Y}$ larger, β will increase, and the shaded area, P_f, will decrease. Thus, P_f is a function of β. The factor β is called the *safety index*.

If Y follows a standard statistical distribution, and if $\overline{Y}$ and σ_Y are known, the probability of failure can be calculated or obtained from statistical tables as a function of the type of distribution and the value of β. Consequently, if Y follows a normal distribution and β is 3.5, then $\overline{Y} = 3.5\sigma_Y$, and, from tables for a *normal distribution*, P_f is 1/9090, or 1.1×10^{-4}.

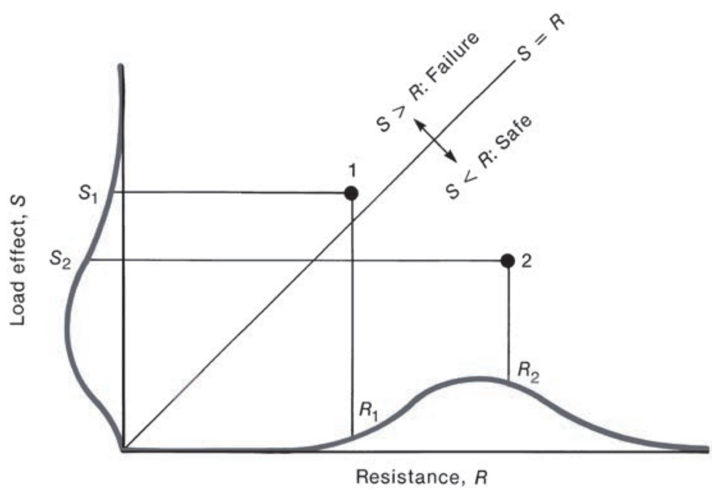


Fig. 2-5
Safe and unsafe combinations of loads and resistances.
(From [2-7].)

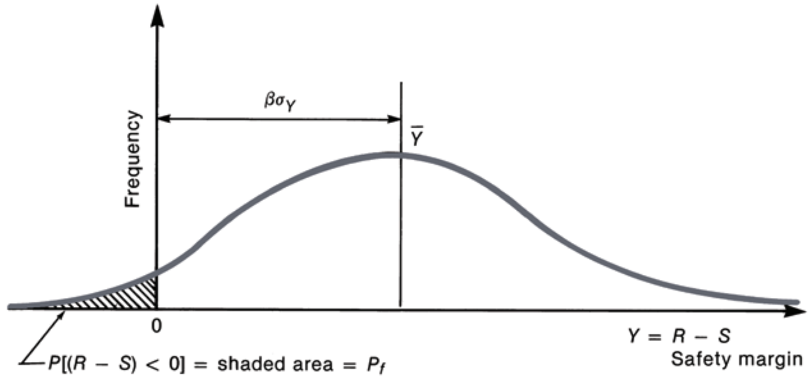


Fig. 2-6
Safety margin, probability of failure, and safety index. (From [2-7].)

This suggests that roughly 1 in every 10,000 structural members designed on the basis that $\beta = 3.5$ will fail due to excessive load or understrength sometime during its lifetime.

The appropriate values of P_f (and hence of β) are chosen by bearing in mind the consequences of failure. Based on current design practice, β is taken between 3 and 3.5 for ductile failures with average consequences of failure and between 3.5 and 4 for sudden failures or failures having serious consequences [2-7], [2-8].

Because the strengths and loads vary independently, it is desirable to have one factor, or a series of factors, to account for the variability in resistances and a second series of factors to account for the variability in load effects. These are referred to, respectively, as *strength-reduction factors* (also called *resistance factors), ϕ*, and *load factors, α*. The resulting design equations are Eqs. (2-2a) through (2-2d).

The derivation of probabilistic equations for calculating values of ϕ and α is summarized and applied in [2-7], [2-8], and [2-9].

The resistance and load factors in the 1971 through 1995 ACI Codes were based on a statistical model, which assumed that if there were a 1/1000 chance of an "overload" and a 1/100 chance of "understrength," the chance that an "overload" and an "understrength" would occur simultaneously is $1/1000 \times 1/100$ or 1×10^{-5}. Thus, the ϕ factors for ductile beams originally were derived so that a strength of ϕR_n would exceed the load effects 99 out of 100 times. The ϕ factors for columns were then divided by 1.1, because the failure of a column has more serious consequences. The ϕ factors for tied columns that fail in a brittle manner were divided by 1.1 a second time to reflect the consequences of the mode of failure. The original derivation is summarized in the appendix of [2-7]. Although this model is simplified by ignoring the overlap in the distributions of R and S in Figs. 2-5 and 2-6, it gives an intuitive estimate of the relative magnitudes of the understrengths and overloads. The 2019 ACI Code [2-10] uses load factors that were modified from those used in the 1995 ACI Code to be consistent with load factors specified in ASCE/SEI 7-16 [2-2] for all types of structures. However, the strength reduction factors were also modified such that the level of safety and the consideration of the consequences of failure have been maintained for consistency with earlier editions of the ACI Code.

2-6 DESIGN PROCEDURES SPECIFIED IN THE ACI BUILDING CODE

Strength Design

In the 2019 ACI Code, design is based on *required strengths* computed from combinations of factored loads and *design strengths* computed as ϕR_n, where ϕ is a *resistance factor*, also known as a *strength-reduction factor, and R_n* is the nominal resistance. This process is called *strength design*. In the AISC Specifications for steel design, the same design process is known as LRFD (Load and Resistance Factor Design). Strength design and LRFD are methods of limit-states design, except that primary attention is placed on the strength limit states, with the serviceability limit states being checked after the original design is completed. Each member design chapter of the 2019 ACI Code contains a general statement that the design strength (defined here as ϕR_n) at all member sections shall equal or exceed the required strength for the load combination listed in Chapter 5 of the Code. Those load combinations will be defined and discussed in the following section.

Reinforced concrete design is usually based on elastic analyses. Cross sections are proportioned to have factored nominal strengths, ϕM_n, ϕP_n, and ϕV_n, greater than or equal to the M_u, P_u, and V_u from an elastic analysis. Because the elastic moments and forces are a statically admissible distribution of forces, and because the resisting-moment diagram is chosen by the designer to be a safe distribution, the strength of the resulting structure is a *lower bound* because the computed failure load is less than or equal to the actual collapse load.

Plastic Design

Plastic design, also referred to as *limit design* (not to be confused with limit-states design) or *capacity design*, is a design process that considers the redistribution of moments as successive cross sections yield, thereby forming *plastic hinges* that lead to a plastic mechanism. These concepts are of considerable importance in seismic design, where the amount of ductility expected from a specific structural system leads to a decrease in the forces that must be resisted by the structure.

2-7 LOAD FACTORS AND LOAD COMBINATIONS IN THE 2019 ACI CODE

The 2019 ACI Code presents load factors and load combinations in Chapter 5, which are from ASCE/SEI 7-16, *Minimum Design Loads for Buildings and Other Structures* [2-2], with slight modifications. The load factors from Code Section 5.3.1 are to be used with the strength-reduction factors in Code Sections 21.2.1 and 21.2.2. These load factors and strength reduction factors were derived in [2-8] for use in the design of steel, timber, masonry, and concrete structures and are used in the AISC LRFD Specification for steel structures [2-11]. For concrete structures, resistance factors that are compatible with the ASCE/SEI 7-16 load factors were derived by ACI Committee 318 with reference to the work of Nowak and Szerszen [2-12].

Terminology and Notation

The ACI Code uses the subscript u to designate the *required strength*, which is a load effect computed from combinations of factored loads. The sum of the combination of factored loads is U as, for example, in

$$U = 1.2D + 1.6L \tag{2-4}$$

where the symbol U and subscript u are used to refer to the sum of the factored loads in terms of loads, or in terms of the effects of the factored loads, M_u, V_u, and P_u.

The member strengths computed using the specified material strengths, f'_c and f_y, and the nominal dimensions, as shown on the drawings, are referred to as the *nominal moment strength*, M_n, or *nominal shear strength*, V_n, and so on. The *reduced nominal strength* or design strength is the nominal strength multiplied by a strength-reduction factor, ϕ. The design equation is thus:

$$\phi M_n \geq M_u \tag{2-2b}$$

$$\phi V_n \geq V_u \tag{2-2c}$$

and so on.

Load Factors and Load Combinations

Load Combinations

Structural failures usually occur under combinations of several loads. In recent years these combinations have been presented in what is referred to as the *companion action format*. This is an attempt to model the expected load combinations.

The load combinations in ACI Code Section 5.3 are examples of *companion action load combinations* chosen to represent realistic load combinations that might occur. In principle, each of these combinations includes one or more *permanent loads* (D or F) with load factors of 1.2, plus the dominant or *principal variable load* (L, S, or others) with a load factor of 1.6, plus one or more *companion-action variable loads*. The companion-action loads are computed by multiplying the specified loads (L, S, W, or others) by *companion-action load factors* between 0.2 and 1.0. The companion-action load factors were chosen to provide results for the companion-action load effects that would be likely during an instance in which the principal variable load is maximized.

In the design of structural members in buildings that are not subjected to significant wind or earthquake forces the factored loads are computed from either Eq. (2-5) or Eq. (2-6):

$$U = 1.4D \tag{2-5}$$

where D is the specified dead load. Where a fluid load, F, is present, it shall be included in accordance with ACI Code Section 5.3.7.

For combinations including dead load; live load, L; and roof loads:

$$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \tag{2-6}$$

where

L = *live load that is a function of use and occupancy*
L_r = *roof live load*
S = *roof snow load*
R = *roof rain load*

If present, lateral earth pressure, H, shall be included in accordance with ACI Code Section 5.3.8.

The terms in Eqs. (2-5) through (2-11) may be expressed as *direct loads* (such as distributed loads from dead and live weight) or *load effects* (such as moments and shears caused by the given loads). The design of a roof structure, or the columns and footings supporting a roof and one or more floors, would take the roof live load equal to the largest of the three loads (L_r or S or R), with the other two roof loads in the brackets taken as zero. For the common case of a member supporting dead and live load only, Eq. (2-6) is written as:

$$U = 1.2D + 1.6L \tag{2-4}$$

If the roof load exceeds the floor live loads, or if a column supports a total roof load that exceeds the total floor live load supported by the column:

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.5W) \tag{2-7}$$

The roof loads are *principal variable loads* in Eq. (2-7), and they are *companion variable loads* in Eqs. (2-6) and (2-8).

$$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \tag{2-8}$$

Wind load, W, is the principal variable load in Eq. (2-8) and is a companion variable load in Eq. (2-7). Wind loads specified in ASCE/SEI 7-16 represent *strength-level* winds, as opposed to the *service-level* wind forces specified in earlier editions of the minimum load standards from ASCE/SEI Committee 7. If the governing building code for the local jurisdiction specifies service-level wind forces, $1.6W$ is to be used in place of $1.0W$ in Eqs. (2-8) and (2-10), and $0.8W$ is to be used in place of $0.5W$ in Eq. (2-7).

Earthquake Loads

If earthquake loads are significant:

$$U = 1.2D + 1.0E + 1.0L + 0.2S \tag{2-9}$$

where the load factor of 1.0 for the earthquake loads corresponds to a *strength-level earthquake*.

Dead Loads That Stabilize Overturning and Sliding

If the effects of dead loads stabilize the structure against wind or earthquake loads,

$$U = 0.9D + 1.0W \tag{2-10}$$

or

$$U = 0.9D + 1.0E \tag{2-11}$$

Load Factor for Small Live Loads

ACI Code Section 5.3.3 allows that the load factor of 1.0 for L in Eqs. (2-7), (2-8), and (2-9) may be reduced to 0.5 except for

- **(a)** parking garages,
- **(b)** areas occupied as places of public assembly, and
- **(c)** all areas where the live load is greater than 100 psf.

Self-Straining Effects

ACI Code Section 5.3.6 uses the letter T to represent actions caused by differential settlement and restrained volume change movements due to either shrinkage or thermal expansion and contraction. Where applicable, these loads are to be considered in combination with other loads. In prior editions of the ACI Code, T was combined with dead load, D, in Eq. (2-6), and thus, the load factor was 1.2. The 2019 edition of the ACI Code states that to establish the appropriate load factor for T the designer is to consider the uncertainty associated with the magnitude of the load, the likelihood that T will occur simultaneously with the maximum value of other applied loads, and the potential adverse effects if the value of T has been underestimated. In any case, the load factor for T is not to be taken less than 1.0. In typical practice, expansion joints and construction pour strips have been used to limit the effects of volume change movements. A recent study of precast structural systems [2-13] gives recommended procedures to account for member and connection stiffnesses and other factors that may influence the magnitude of forces induced by volume change movements.

In the analysis of a building frame, it is frequently best to analyze the structure elastically for each load to be considered and to combine the resulting moments, shears, and so on for each member according to Eqs. (2-4) to (2-11). (Exceptions to this are analyses of cases in which linear superposition does not apply, such as second-order analyses of frames. These must be carried out at the factored-load level.) The procedure used is illustrated in Example 2-1.

EXAMPLE 2-1 Computation of Factored-Load Effects

Figure 2-7 shows a beam and column from a concrete building frame. The loads on the beam are a dead load, $D = 1.58$ kips/ft, and a live load, $L = 0.75$ kip/ft. Additionally, a wind load is represented by the concentrated loads at the joints. Typical moments in the beam and the columns over and under the beam due to $1.0D$, $1.0L$, and $1.0W$ are shown in Figs. 2-7b to 2-7d.

Compute the required strengths, using Eqs. (2-4) through (2-11). For the moment at beam section A, four load cases must be considered:

(a) $U = 1.4D$ (2-5)

- Because there are no fluid or thermal forces to consider, $U = 1.4 \times -39 = -54.6$ k-ft.

(b) $U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$ (2-6)

- Assuming that there is no differential settlement of the interior columns relative to the exterior columns and assuming there is no restrained shrinkage, the self-equilibrating actions, T, will be taken to be zero.
- Because the beam being considered is not a roof beam, L_r, S, and R are all equal to zero. (Note that the axial loads in the columns will include axial forces from the roof load and the slab load.)

Equation (2-6) becomes

$$\begin{aligned} U &= 1.2D + 1.6L \qquad (2\text{-}4) \\ &= 1.2 \times -39 + 1.6 \times -19 = -77.2 \text{ k-ft} \end{aligned}$$

(c) Equation (2-7) does not govern because this is not a roof beam.

(d) For Eq. (2-8), assume strength-level wind forces have been specified, so the load factor of 1.0 is used for W.

$$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \qquad (2\text{-}8)$$

where ACI Code Section 5.3.3 normally allows $1.0L$ to be reduced to $0.5L$, so,

$$\begin{aligned} U &= 1.2D + 1.0W + 0.5L \\ &= 1.2 \times -39 \pm 1.0 \times 134 + 0.5 \times -19 \\ &= -56.3 \pm 134 \\ &= -190 \text{ or } +77.7 \text{ k-ft} \end{aligned}$$

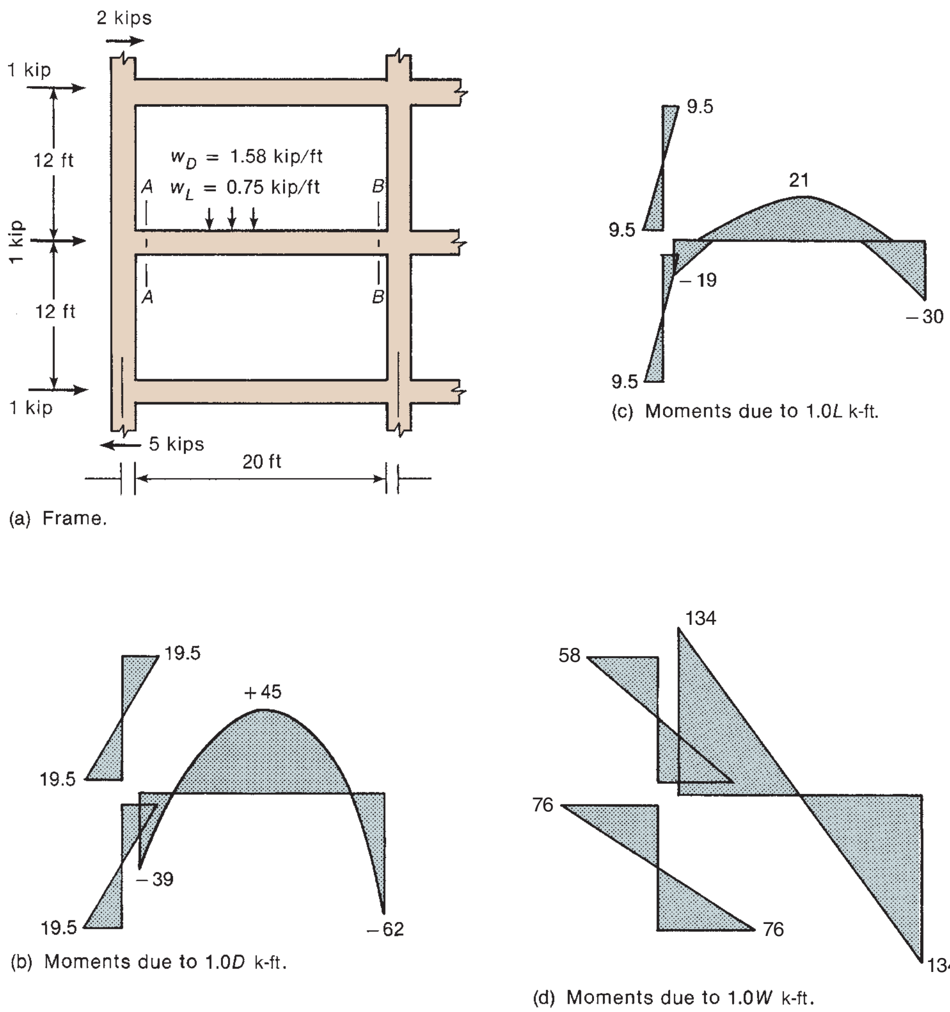


Fig. 2-7
Moment diagrams—
Example 2-1.

The positive and negative values of the wind-load moment are due to the possibility of winds alternately blowing on the two sides of the building.

(e) The dead-load moments can counteract a portion of the wind- and live-load moments. This makes it necessary to consider Eq. (2-10):

$$
\begin{aligned}
U &= 0.9D + 1.6W \qquad (2\text{-}10)\\
&= 0.9 \times -39 \pm 1.0 \times 134 = -35.1 \pm 134\\
&= +98.9 \text{ or } -169 \text{ k-ft}
\end{aligned}
$$

Thus the required strengths, M_u, at section A–A are +98.9 k-ft and −190 k-ft. ■

This type of computation is repeated for a sufficient number of sections to make it possible to draw shear-force and bending-moment envelopes for the beam.

Strength-Reduction Factors, ϕ

ACI Code Chapter 21 defines the following set of strength-reduction factors to be used in conjunction with the load combinations given in ACI Code Eqs. (5.3.1a) through (5.3.1g).

Flexure or Combined Flexure and Axial Load

Tension-controlled sections	$\phi = 0.90$
Compression-controlled sections:	
(a) Members with spiral reinforcement	$\phi = 0.75$
(b) Other compression-controlled sections	$\phi = 0.65$

There is a transition region between tension-controlled and compression-controlled sections. The concept of tension-controlled and compression-controlled sections, and the resulting strength-reduction factors, will be presented for beams in flexure, axially loaded columns, and columns loaded in combined axial load and bending in Chapters 4, 5, and 11. The derivation of the ϕ factors will be introduced at that time.

Other actions

Shear and torsion	$\phi = 0.75$
Bearing on concrete	$\phi = 0.65$
Strut-and-tie model	$\phi = 0.75$

2-8 LOADINGS AND ACTIONS

Direct and Indirect Actions

An *action* is anything that gives rise to stresses in a structure. The term *load* or *direct action* refers to concentrated or distributed forces resulting from the weight of the structure and its contents, or pressures due to wind, water, or soil. An *indirect action* or *imposed deformation* is a movement or deformation that does not result from applied loads, but causes stresses in a structure. Examples are ground motions during an earthquake, uneven support settlements, and shrinkage of concrete if it is not free to shorten.

Concrete shrinkage normally results in internal stresses that are *self-equilibrating*. Consider, for example, a prism of concrete with a reinforcing bar along its axis. As the concrete shrinks, its shortening is resisted by the reinforcement. As a result, a compressive force develops in the steel and an equal and opposite tensile force develops in the concrete, as shown in Fig. 2-8. If the concrete cracks from this tension, the tensile force in the concrete at the crack is zero, and for equilibrium, the steel force must also disappear at the cracked section. Section 1.3.3 of ASCE/SEI 7-16 refers to imposed deformations as *self-straining forces*.

Classifications of Loads

Loads may be described by their variability with respect to time and location. A *permanent* load remains roughly constant once the structure is completed. Examples are the self-weight of the structure and soil pressure against foundations. *Variable* loads, such as occupancy loads and wind loads, change from time to time. Variable loads may be *sustained loads* of long duration, such as the weight of filing cabinets in an office, or loads of *short duration*, such as the weight of people in the same office. Creep deformations of concrete structures result from permanent loads and the sustained portion of the variable loads. A third category is *accidental loads*, which include vehicular collisions and explosions.

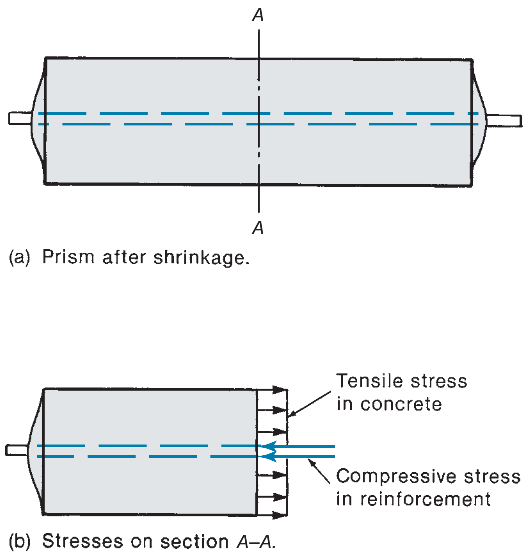


Fig. 2-8
Self-equilibrating stresses due to shrinkage.

Variable loads may be *fixed* or *free* in location. Thus, the live loading in an office building is free, because it can occur at any point in the loaded area. A train load on a bridge is not fixed longitudinally, but is fixed laterally by the rails.

Loads frequently are classified as *static loads* if they do not cause any appreciable acceleration or vibration of the structure or structural elements and as *dynamic loads* if they do. Small accelerations are often taken into account by increasing the specified static loads to account for the increases in stress due to such accelerations and vibrations. Larger accelerations, such as those which might occur in highway bridges, crane rails, or elevator supports are accounted for by multiplying the effect of the live load by an *impact factor*. Alternatively, dynamic analysis may be used.

Three levels of live load or wind load may be of importance. The load used in calculations involving the strength limit states should represent the maximum load on the structure in its lifetime. Wherever possible, therefore, the specified live, snow, and wind loadings should represent the mean value of the corresponding maximum lifetime load. A *companion-action load* is the portion of a variable load that is present on a structure when some other variable load is at its maximum. In checking the serviceability limit states, it may be desirable to use a *frequent* live load, which is some fraction of the mean maximum lifetime load (generally, 50 to 60 percent); for estimating sustained load deflections, it may be desirable to consider a *sustained* or *quasi-permanent* live load, which is generally between 20 and 30 percent of the specified live load.

Loading Specifications

Most cities in the United States base their building codes on the *International Building Code* [2-14]. The loadings specified in this code are based on the loads recommended in *Minimum Design Loads for Buildings and Other Structures,* ASCE/SEI 7-16 [2-2].

In the following sections, the types of loadings presented in ASCE/SEI 7-16 will be briefly reviewed. This review is intended to describe the characteristics of the various loads. For specific values, the reader should consult the building code in effect in his or her own locality.

Dead Loads

The *dead load* on a structural element is the weight of the member itself, plus the weights of all materials permanently incorporated into the structure and supported by the member in question. This includes the weights of permanent partitions or walls, the weights of plumbing stacks, electrical feeders, permanent mechanical equipment, and so on. Tables of dead loads are given in ASCE/SEI 7-16.

In the design of a reinforced concrete member, it is necessary to estimate the weight of the member. Methods of making this estimate for beams and one-way slabs are given in Chapter 5. Once the member size has been computed, its weight is calculated by multiplying the volume by the density of concrete, taken as 145 lb/ft^3 for plain concrete and 150 lb/ft^3 for reinforced concrete (5 lb/ft^3 is added to account for reinforcement). For lightweight concrete members, the density of the concrete must be determined from trial batches or as specified by the producer. In heavily reinforced members, the density of the reinforced concrete may exceed 150 lb/ft^3.

In working with SI units (metric units), the weight of a member is calculated by multiplying the volume by the mass density of concrete and the gravitational constant, 9.81 N/kg. In this calculation, it is customary to take the mass density of normal-density concrete containing an average amount of reinforcement (roughly 2 percent by volume) as 2450 kg/m^3, made up of 2300 kg/m^3 for the concrete and 150 kg/m^3 for the reinforcement. The weight of a cubic meter of reinforced concrete is thus $(1\ \text{m}^3 \times 2450\ \text{kg/m}^3 \times 9.81\ \text{N/kg})/1000 =$ 24.0 kN, and its weight density is 24 kN/m^3.

The dead load referred to in Eqs. (2-5) to (2-11) is the load computed from the dimensions shown on drawings and the assumed densities. It is therefore close to the mean value of this load. Actual dead loads will vary from the calculated values, because the actual dimensions and densities may differ from those used in the calculations. Sometimes the materials for the roof, partitions, or walls are chosen on the basis of a separate bid document, and their actual weights may be unknown at the time of the design. Tabulated densities of materials frequently tend to underestimate the actual dead loads of the material in place in a structure.

Some types of dead load tend to be highly uncertain. These include pavement on bridges, which may be paved and repaved several times over a period of time, or where a greater thickness of pavement may be applied to correct sag or alignment problems. Similarly, earth fill over an underground structure may be up to several inches thicker than assumed and may or may not be saturated with water. In the construction of thin curved-shell roofs or other lightweight roofs, the concrete thickness may exceed the design values and the roofing may be heavier than assumed.

If dead-load moments, forces, or stresses tend to counteract those due to live loads or wind loads, the designer should carefully examine whether the counteracting dead load will always exist. Thus, dead loads due to soil or machinery may be applied late in the construction process and may not be applied evenly to all parts of the structure at the same time, leading to a potentially critical set of moments, forces, or stresses under partial loads.

Live Loads Due to Use and Occupancy

Most building codes contain a table of design or specified live loads. To simplify the calculations, these are expressed as uniform loads on the floor area. In general, a building live load consists of a sustained portion due to day-to-day use (see Fig. 2-4) and a variable portion generated by unusual events. The sustained portion changes a number of times during the life of the building—when tenants change, when the offices are rearranged, and so on. Occasionally, high concentrations of live loading occur during periods when adjacent spaces are remodeled, when office parties are held, or when material is stored temporarily. The loading given in building codes is intended to represent the maximum sum of these loads that will occur on a small area during the life of the building. Typical specified live loads are given in Table 2-1.

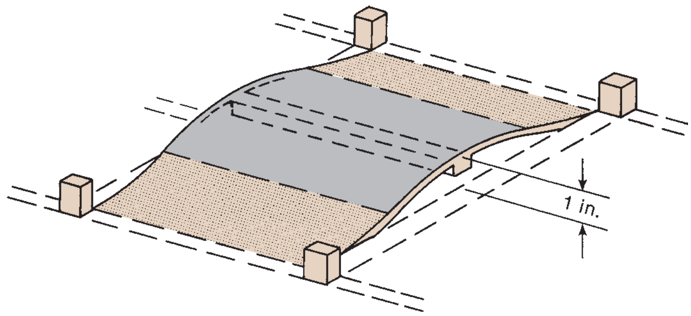
In buildings where nonpermanent partitions might be erected or rearranged during the life of the building, allowance should be made for the weight of these partitions. ASCE/SEI 7-16 specifies that provision for partition weight should be made, regardless of whether partitions are shown on the plans, unless the specified live load exceeds 80 psf. It is customary to represent the partition weight with a uniform load of 20 psf or a uniform load computed from the actual or anticipated weights of the partitions placed in any probable position. ASCE/SEI 7-16 considers this to be live load, because it may or may not be present.

As the loaded area increases, the average maximum lifetime load decreases because, although it is quite possible to have a heavy load on a small area, it is unlikely that this would occur in a large area. This is taken into account by multiplying the specified live loads by a *live-load reduction factor*.

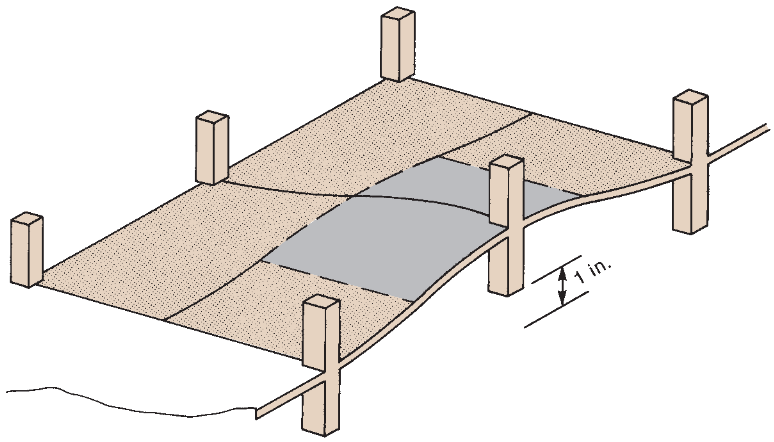
In ASCE/SEI 7-16, this factor is based on the *influence area*, A_I, for the member being designed. The concept of influence lines and influence areas is discussed in Chapter 5. To figure out the influence area of a given member, one can imagine that the member in question is raised by a unit amount, say, 1 in. as shown in Fig. 2-9. The portion of the loaded area that is raised when this is done is called the *influence area*, A_I, because loads acting anywhere in this area will have a significant impact on the load effects in the

TABLE 2-1 Typical Live Loads Specified in ASCE/SEI 7-16

	Uniform, psf	Concentration, lb
Apartment buildings		
Private rooms and corridors serving them	40	
Public rooms and corridors serving them	100	
Office buildings		
Lobbies and first-floor corridors	100	2000
Offices	50	2000
Corridors above first floor	80	2000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Schools		
Classrooms	40	1000
Corridors above first floor	80	1000
First-floor corridors	100	1000
Stairs and exitways	100	
Storage warehouses		
Light	125	
Heavy	250	
Stores		
Retail		
Ground floor	100	1000
Upper floors	75	1000
Wholesale, all floors	125	1000



(a) Interior floor beam.



(b) Edge column.

Fig. 2-9
Influence areas.

member in question. This concept is illustrated in Fig. 2-9 for an interior floor beam and an edge column.

In contrast, the *tributary area*, A_T, extends out from the beam or column to the lines of zero shear in the floor around the member under consideration. For the beam in Fig. 2-9a, the limits on A_T are given by the dashed lines halfway to the next beam on each side. The tributary areas are shown in a darker shading in Figs. 2-9a and 2-9b. An examination of Fig. 2-9a shows that A_T is half of A_I for an interior beam. For the column in Fig. 2-9b, A_T is one-fourth of A_I. Because two-way slab design is based on the total moments in one slab panel, the influence area for such a slab is defined by ASCE/SEI 7-16 as the total panel area.

Previous versions of the ASCE/SEI 7 document allowed the use of reduced live loads, L, in the design of members, based on the influence area A_I. However, the influence-area concept is not widely known compared with that of the tributary area, A_T. In ASCE/SEI 7-16, the influence area is given as $A_I = K_{LL}A_T$, where A_T is the tributary area of the member being designed and K_{LL} is the ratio A_I/A_T. The reduced live load, L, is given by

$$L = L_o\left[0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right] \tag{2-12}$$

where L_o is the unreduced live load. Values of K_{LL} are given as follows:

Interior columns and exterior columns without cantilever slabs	$K_{LL} = 4$
Exterior columns with cantilever slabs	$K_{LL} = 3$
Corner columns with cantilever slabs	$K_{LL} = 2$
Interior beams and edge beams without cantilever slabs	$K_{LL} = 2$
All other members, including one-way and two-way slabs	$K_{LL} = 1$

The live-load reduction applies only to live loads due to use and occupancy (not for snow, etc.). No reduction is made for areas used as places of public assembly, for garages, or for roofs. In ASCE/SEI 7-16, the reduced live load cannot be less than 50 percent of the unreduced live load for columns supporting one floor or for flexural members, and no less than 40 percent for other members.

For live loads exceeding 100 psf, no reduction is allowed by ASCE/SEI 7-16, except that the design live load on columns supporting more than one floor can be reduced by 20 percent.

The reduced uniform live loads are then applied to those spans or parts of spans that will give the maximum shears, moments, and so on, at each critical section. This approach is illustrated in Chapter 5.

The ASCE/SEI 7-16 standard requires that office and garage floors and sidewalks be designed to safely support either the reduced uniform design loads or a concentrated load of from 1000 to 8000 lb (depending on occupancy), spread over an area of from 4.5 in. by 4.5 in. to 30 in. by 30 in. The concentrated loads are intended to represent heavy items such as office safes, pianos, car wheels, and so on.

In checking the concentrated load capacity, it generally is necessary to assume an effective width of floor to carry the load to the supports. For one-way floors, this is usually the width of the concentrated load reaction plus one slab effective depth on each side of the load. For two-way slabs, Chapter 13 shows that a concentrated load applied at various points in the slab gives maximum moments (at midspan and near the support columns) that are similar in magnitude to those computed for a complete panel loaded with a uniform load. In many cases, this makes it unnecessary to check the concentrated load effects on maximum moment for two-way slabs.

The live loads are assumed to be large enough to account for the impact effects of normal use and traffic. Special impact factors are given in the loading specifications for supports of elevator machinery, large reciprocating or rotating machines, and cranes.

Classification of Buildings for Wind, Snow, and Earthquake Loads

The ASCE/SEI 7-16 requirements for design for wind, snow, and earthquake become progressively more restrictive as the level of risk to human life in the event of a collapse increases. These are referred to as *risk categories*:

I. Buildings and other structures that represent a low hazard to human life in the event of failure, such as agricultural facilities.

II. Buildings and other structures that do not fall into categories I, III, or IV.

III. Buildings or other structures that represent a substantial hazard to human life in the event of failure, such as assembly occupancies, schools, and detention facilities. Also, buildings and other structures not included in risk category IV that contain a sufficient quantity of highly toxic or explosive substances that pose a significant threat to the general public if released.

IV. Buildings and other structures designated as essential facilities, such as hospitals, fire and police stations, communication centers, and power-generating stations and facilities. Also, buildings and other structures that contain a sufficient quantity of highly toxic or explosive substances that pose a significant threat to the general public if released.

Snow Loads, *S*

Snow accumulation on roofs is influenced by climatic factors, roof geometry, and the exposure of the roof to wind. Unbalanced snow loads due to drifting or sliding of snow or uneven removal of snow by workers are very common. Large accumulations of snow often will occur adjacent to parapets or other points where roof heights change. ASCE/SEI 7-16 gives detailed rules for calculating snow loads to account for the effects of snow drifts. It is necessary to design for either a uniform or an unbalanced snow load, whichever gives the worst effect.

Roof Live Loads, L_r, and Rain Loads, *R*

In addition to snow loads, roofs should be designed for certain minimum live loads (L_r) to account for workers or construction materials on the roof during erection or when repairs are made. Consideration must also be given to loads due to rainwater, *R*. Because roof drains are rarely inspected to remove leaves or other debris, ASCE/SEI 7-16 requires that roofs be able to support the load of all rainwater that could accumulate on a particular portion of a roof if the primary roof drains were blocked. Frequently, controlled-flow roof drains are used to slow the flow of rainwater off a roof. This reduces plumbing and storm sewage costs but adds to the costs of the roof structure.

If the design snow load is small and the roof span is longer than about 25 ft, rainwater will tend to form ponds in the areas of maximum deflection. The weight of the water in these regions may cause an increase in the deflections, allowing more water to collect, and so on. If the roof is not sufficiently stiff, a *ponding failure* will occur when the weight of ponded water reaches the capacity of the roof members [2-15].

Construction Loads

During the construction of concrete buildings, the weight of the fresh concrete is supported by formwork, which frequently are supported by floors lower down in the structure. In addition, construction materials are often piled on floors or roofs during construction. ACI Code Section 26.11.2.1(c) states the following:

> No construction loads shall be placed on, nor any formwork removed from, any part of the structure under construction except when that portion of the structure in combination with remaining formwork has sufficient strength to support safely its weight and loads placed thereon and without impairing serviceability.

Wind Loads

The pressure exerted by the wind is related to the square of its velocity. Due to the roughness of the earth's surface, the wind velocity at any particular instant consists of an average velocity plus superimposed turbulence, referred to as *gusts*. As a result, a structure subjected to wind loads assumes an average deflected position due to the average velocity pressure and vibrates from this position in response to the gust pressure. In addition, there will generally be deflections transverse to the wind (due to vortex shedding) as the wind passes the building. The vibrations due to the wind gusts are a function of (1) the relationship between the natural energy of the wind gusts and the energy necessary to displace the building, (2) the relationship between the gust frequencies and the natural frequency of the building, and (3) the damping of the building [2-16].

Three procedures are specified in ASCE/SEI 7-16 for the calculation of wind pressures on buildings: the *envelope procedure*, limited in application to buildings with a mean roof height of 60 ft or less; the *directional procedure*, limited to regular buildings that do not have response characteristics making it subject to a cross-wind loading, vortex shedding, or channeling of the wind due to upwind obstructions; and the *wind tunnel procedure*, used for complex buildings. We shall consider the directional procedure. Variations of this method apply to design of the main wind-force-resisting systems of buildings and to the design of components and cladding.

In the directional procedure, the wind pressure on the main wind-force-resisting system is

$$p = qGC_p - q_i(GC_{pi}) \tag{2-13}$$

where either $q = q_z$, the velocity pressure evaluated at height z above the ground on the windward wall, or $q = q_h$, the pressure on the roof, leeward walls, and sidewalls, evaluated at the mean roof height, h, and q_i is the internal pressure or suction on the interior of the walls and roof of the building, also evaluated at the mean roof height.

The total wind pressure p, is the sum of the external pressure on the windward wall and the suction on the leeward wall, which is given by the first term on the right-hand side of Eq. (2-13) plus the second term, p_i, which accounts for the internal pressure. The internal pressure, p_i, is the same on all internal surfaces at any given time. Thus, the internal pressure or suction on the inside of the windward wall is equal but opposite in direction to the internal pressure or suction on the inside of the leeward wall. As a result, the interior wind forces on opposite walls cancel out in most cases, leaving only the external pressure to be resisted by the main wind-force-resisting system. The terms in Eq. (2-13) are defined as:

1. Design pressure, *p*. The *design pressure* is an equivalent static pressure or suction in psf assumed to act perpendicular to the surface in question. On some surfaces, it varies over the height; on others, it is assumed to be constant.

2. Wind Velocity pressure, *q*. The *wind velocity pressure at height z on the windward wall,* q_z, is the pressure (psf) exerted by the wind on a flat plate suspended in the wind stream. It is calculated as

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \tag{2-14}$$

where

V = nominal design 3-sec gust wind speed in miles per hour at a height of 33 ft (10 m) above the ground in Exposure C, open terrain (3 percent probability of exceedance in 50 years; Category III and IV buildings)

K_z = velocity pressure exposure coefficient, which increases with height above the surface and reflects the roughness of the surface terrain

K_{zt} = the topographic factor that accounts for increases in wind speed as it passes over hills

K_d = directionality factor equal to 0.85 for rectangular buildings and 0.90 to 0.95 for circular tanks and the like

The constant 0.00256 reflects the mass density of the air and accounts for the mixture of units in Eq. (2-14).

At any location, the mean wind velocity is affected by the roughness of the terrain upwind from the structure in question. At a height of 700 to 1500 ft, the wind reaches a steady velocity, as shown by the vertical lines in the plots of K_z in Fig. 2-10. Below this height, the velocity decreases and the turbulence, or gustiness, increases as one approaches the surface. These effects are greater in urban areas than in rural areas, due to the greater surface roughness in built-up areas. The factor K_z in Eq. (2-14) relates the wind pressure at any elevation z feet to that at 33 ft (10 m) above the surface for Exposure C. ASCE/SEI 7-16 gives tables and equations for K_z as a function of the type of exposure (urban, country, etc.) and the height above the surface.

For **side walls, leeward wall**, and **roof surfaces**, q_h is a constant suction (negative pressure) on side walls, the leeward wall, and flat roofs, evaluated by using h equal to the average height of the roof. For a sloping roof, q_h must be evaluated as both a positive (downward) pressure and a negative (upward) pressure for the portion of the roof sloping toward the direction of the wind.

3. Gust-effect factor, *G*. The gust-effect factor, G, in Eq. (2-13) relates the dynamic properties of the wind and the structure. For flexible buildings, it is calculated. For most buildings that tend to be stiff, it is taken to be equal to 0.85.

4. External pressure coefficient, C_p. When wind blows past a structure, it exerts a positive pressure on the windward wall and a negative pressure (suction) on the leeward wall, side walls, and roof as shown in Fig. 2-11. The overall pressures to be used in the design of a structural frame are computed via Eq. (2-13), where C_p is the sum or difference in the pressure coefficients for the windward and leeward walls. Thus, $C_p = +0.80$ (pressure) on the left-hand (windward) wall in Fig. 2-11 and $C_p = -0.50$ (suction) on the right-hand (leeward) wall add together to produce the load on the frame because they have the same direction. Values of the pressure coefficients are given in the ASCE/SEI 7-16. Typical values are shown in Fig. 2-11 for a building having the shape and proportions shown. For a rectangular building with the wind on the narrow side, C_p for the leeward wall varies between -0.5 and -0.2.

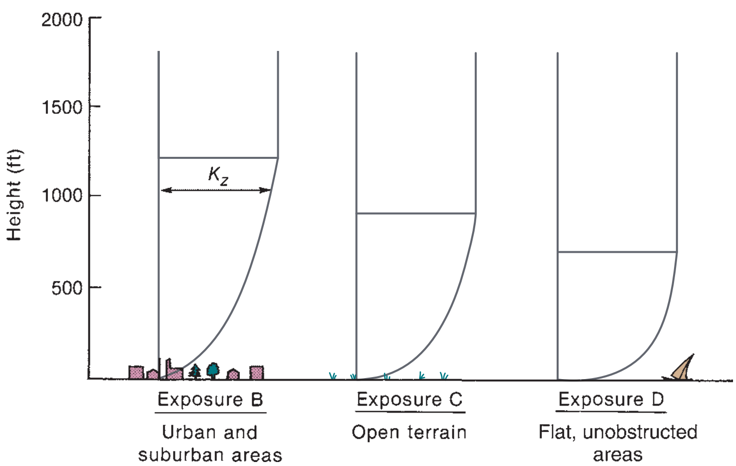


Fig. 2-10
Profiles of velocity pressure exposure coefficient, K_z, for differing terrain.