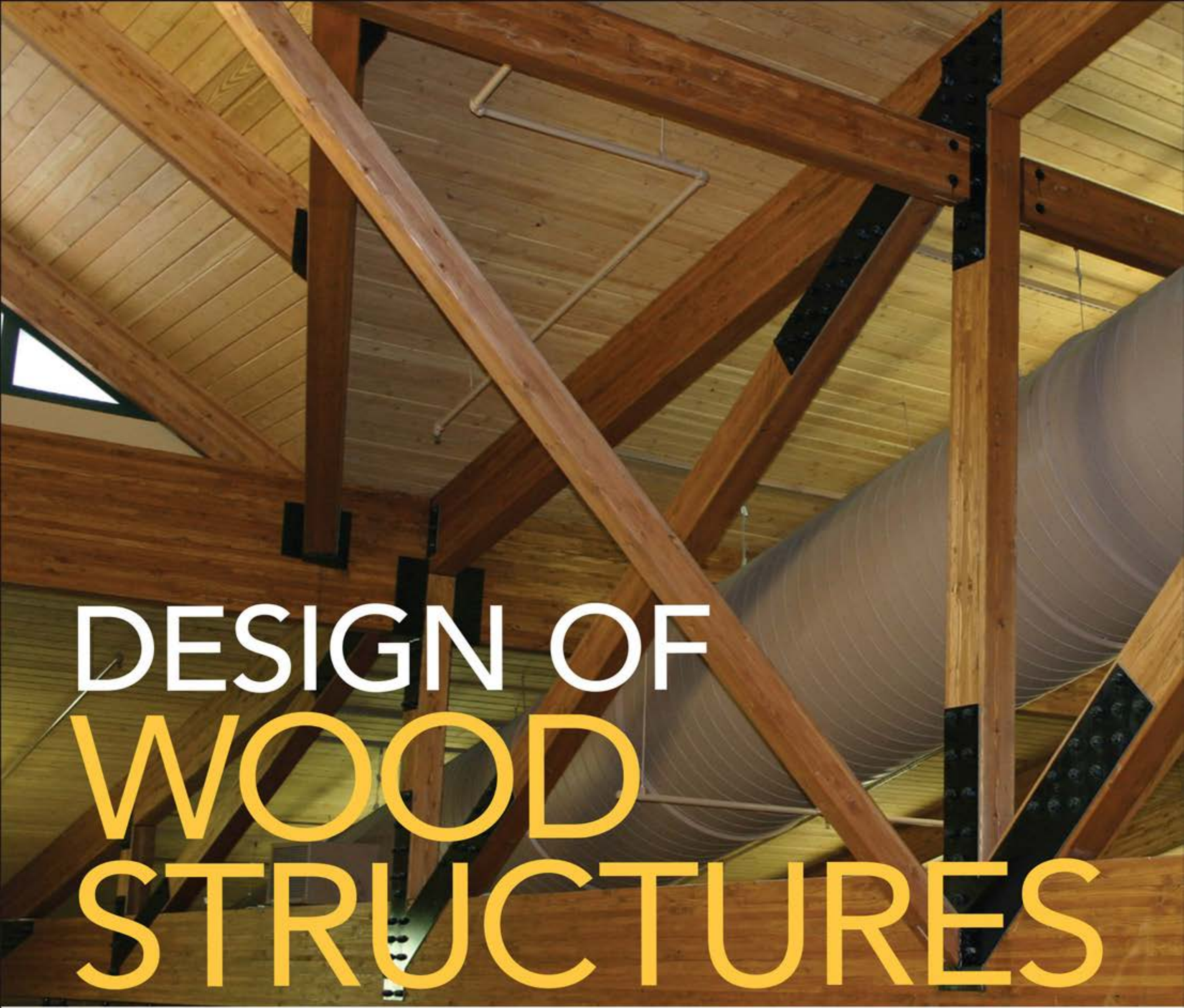

DESIGN OF WOOD STRUCTURES

ASD/LRFD

Eighth Edition

Donald E. Breyer
Kelly Cobeen
Zeno Martin

Mc Graw Hill

ICC INTERNATIONAL CODE COUNCIL®

Design of Wood Structures—ASD/LRFD

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Design of Wood Structures—ASD/LRFD

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Version 1.0

To our families:
Matthew, Kerry, Daniel, and Sarah
Mom, Dad, and Matthew
Patti, Kris, and Pam

Contents

Preface

The purpose of this book is to introduce engineers, technologists, and architects to the design of wood structures. It is intended to serve either as a text for a course in timber design or as a reference for systematic self-study of the subject.

The book will lead the reader through the complete design of a wood structure (except for the foundation). The sequence of the material follows the same general order that it would in actual design:

1. Vertical design loads and lateral forces.
2. Design for vertical loads (beams and columns).
3. Design for lateral forces (horizontal diaphragms and shearwalls).
4. Connection design (including the overall tying together of the vertical- and lateral-force-resisting systems).

The need for such an overall approach to the subject became clear from experience gained in teaching timber design at the undergraduate and graduate levels.

This text pulls together the design of the various elements into a single reference. A large number of practical design examples are provided throughout the text. Because of their widespread usage, buildings naturally form the basis of the majority of these examples. However, the principles of member design and diaphragm design have application to other structures (such as concrete formwork and falsework).

This book relies on practical, current industry literature as the basis for structural design. This includes publications of the American Wood Council (AWC), the International Code Council (ICC), the American Society of Civil Engineers (ASCE), APA—The Engineered Wood Association, and the American Institute of Timber Construction (AITC).

In the writing of this text, an effort has been made to conform to the spirit and intent of the reference documents. The interpretations are those of the authors and are intended to reflect current structural design practice. The material presented is suggested as a guide only, and final design responsibility lies with the structural engineer.

The eighth edition of this book updates it to be consistent with the 2018 International Building Code and its relevant reference standards that include but are not limited to:

1. The 2018 *National Design Specification for Wood Construction* (NDS).
2. The 2015 *Special Design Provisions for Wind and Seismic* (SDPWS)
3. The 2016 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16).

Questions or comments about the text or examples may be addressed to any of the authors. Direct any correspondence to:

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Acknowledgment and appreciation for help in writing this text are given to our numerous colleagues in the wood design profession. Suggestions and information were obtained from many engineers and suppliers, and their help is gratefully recognized.

Nomenclature

Organizations

AITC
American Institute of Timber Construction
P.O. Box 23145
Portland, OR 97281
www.aitc-glulam.org

ALSC
American Lumber Standard Committee, Inc.
P.O. Box 210
Germantown, MD 20875-0210
www.alsc.org
APA—The Engineered Wood Association
P.O. Box 11700
Tacoma, WA 98411-0700
www.apawood.org

ASCE
American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191
www.asce.org

ATC
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, CA 94065
www.atcouncil.org

AWC
American Wood Council
222 Catoctin Circle, SE
Leesburg, VA 20175
www.awc.org

AWPA
American Wood Protection Association
P.O. Box 361784
Birmingham, AL 35236
www.awpa.com

BSSC
Building Seismic Safety Council
National Institute of Building Sciences
1090 Vermont Avenue NW, Suite 700
Washington, DC 20005
www.bssconline.org

CANPLY
Canadian Plywood Association
735 West 15 Street
North Vancouver, British Columbia,
Canada V7M 1T2
www.canply.org

CPA
Composite Panel Association
19465 Deerfield Ave., Suite 306
Leesburg, VA 20176
www.compositepanel.org

CWC
Canadian Wood Council
99 Bank Street, Suite 400
Ottawa, Ontario, Canada K1P 6B9
www.cwc.ca

FPL
U.S. Forest Products Laboratory
USDA Forest Service
One Gifford Pinchot Drive
Madison, WI 53726-2398
www.fpl.fs.fed.us

ICC
International Code Council
500 New Jersey Avenue NW
Washington, DC 20001
www.iccsafe.org

ISANTA
International Staple, Nail and Tool Association
512 West Burlington Avenue, Suite 203
La Grange, IL 60525-2245
www.isanta.org

MSRLPC
MSR Lumber Producers Council
6300 Enterprise Lane
Madison, WI 53719
www.msrlumber.org

NELMA
Northeastern Lumber Manufacturers Association
272 Tuttle Road
P.O. Box 87A
Cumberland Center, ME 04021
www.nelma.org

NFBA
National Frame Building Association
8735 W. Higgins Rd., Suite 300
Chicago, IL 60631
www.nfba.org

NHLA
National Hardwood Lumber Association
P.O. Box 34518
Memphis, TN 38184-0518
www.nhla.com

NLGA
National Lumber Grades Authority
13401 108th Ave., Suite 105
Surrey, British Columbia, Canada V3T 5T3
www.nlga.org

NSLB
Northern Softwood Lumber Bureau
272 Tuttle Road
P.O. Box 87A
Cumberland Center, ME 04021
www.nelma.org

PLIB
Pacific Lumber Inspection Bureau
1010 S. 336th St., Suite 300
Federal Way, WA 98003-6214
www.plib.org

SBCA
Structural Building Components Association
6300 Enterprise Lane
Madison, WI 53719
www.sbcindustry.com

SEAOC
Structural Engineers Association of California
1400 K Street, Suite 212
Sacramento, CA 95814
www.seaoc.org

SFPA
Southern Forest Products Association
6660 Riverside Dr., Suite 212
Metairie, LA 70003
www.sfpa.org
www.southernpine.com

SLMA
Southeastern Lumber Manufacturers Association
200 Greencastle Road
Tyrone, GA 30290
www.slma.org

SPIB
Southern Pine Inspection Bureau, Inc.
4709 Scenic Highway
Pensacola, FL 32504-9094
www.spib.org

TPI
Truss Plate Institute
218 N. Lee Street, Suite 312
Alexandria, VA 22314
www.tpinst.org

WCLIB
West Coast Lumber Inspection Bureau
P.O. Box 23145
Portland, OR 97281-3145
www.wclib.org

WIJMA
Wood I-Joist Manufacturing Association
200 East Mallard Drive
Boise, ID 83706
www.i-joist.org

WRCLA
Western Red Cedar Lumber Association
1501-700 West Pender Street
Vancouver, British Columbia, Canada
V6C 1G8
www.realcedar.com

WWPA
Western Wood Products Association
522 Southwest Fifth Avenue, Suite 500
Portland, OR 97204-2122
www.wwpa.org

Publications

ASCE 7:	American Society of Civil Engineers (ASCE). 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7-16), ASCE, Reston, VA.
ASD/LRFD Manual:	American Wood Council (AWC). 2018. *ASD/LRFD Manual for Engineered Wood Construction*, 2018 ed., AWC, Leesburg, VA.
IBC:	International Code Council (ICC). 2018. *International Building Code* (IBC), 2018 ed., ICC, Washington, DC.
NDS:	American Wood Council (AWC). 2018. *National Design Specification for Wood Construction* (NDS), ANSI/AWC NDS-2018, AWC, Leesburg, VA.
SDPWS:	American Wood Council (AWC). 2015. *Special Design Provisions for Wind and Seismic* (SDPWS), AWC, Leesburg, VA.
TCM:	American Institute of Timber Construction (AITC). 2012. *Timber Construction Manual*, 6th ed., John Wiley & Sons Inc., Hoboken, NJ.

Additional publications are given at the end of each chapter.

Units

ft	foot, feet
ft^2	square foot, square feet
in.	inch, inches
$in.^2$	square inch, square inches
k	1000 lb (kip, kilopound)
ksi	kips per square inch ($k/in.^2$)
mph	miles per hour
pcf	pounds per cubic foot (lb/ft^3)
plf	pounds per lineal foot (lb/ft)
psf	pounds per square foot (lb/ft^2)
psi	pounds per square inch ($lb/in.^2$)
sec	second

CHAPTER 1

Wood Buildings and Design Criteria

1.1 Introduction

There are probably more buildings constructed with wood than any other structural material. Many of these buildings are single-family residences, but many larger apartment buildings as well as commercial and industrial buildings also use wood framing. Wood is currently moving into mid-rise construction, with buildings of four to six stories being increasingly common. In addition, while the majority of wood-frame construction has in the past been light-frame systems, cross-laminated timber (CLT) systems are now emerging.

The widespread use of wood in the construction of buildings has both an economic and an aesthetic bases. The ability to construct wood buildings with a minimal amount of specialized equipment has kept the cost of wood-frame buildings competitive with other types of construction. On the other hand, where architectural considerations are important, the beauty and the warmth of exposed wood are difficult to match with other materials.

Wood-frame construction has evolved from a method used in primitive shelters into a major field of structural design. However, in comparison with the time devoted to steel and reinforced concrete design, timber design is not given sufficient attention in most colleges and universities.

This book is designed to introduce the subject of timber design as applied to wood light-frame building construction. Although the discussion centers on building design, the concepts also apply to the design of other types of wood-frame structures. Final responsibility for the design of a building rests with the structural engineer. However, this book is written to introduce the subject to a broad audience. This includes engineers, engineering technologists, architects, and others concerned with building design. A background in statics and strength of materials is required to adequately follow the text. Most wood-frame buildings are highly redundant structures, but for design simplicity they are assumed to be made up of statically determinate members. The ability to analyze simple trusses, beams, and frames is also necessary.

1.2 Types of Buildings

There are various types of framing systems that can be used in wood buildings. The most common type of wood-frame construction uses a system of horizontal diaphragms and vertical shearwalls to resist lateral forces, and this book deals specifically with the design of this basic type of building. At one time building codes classified a shearwall building as a *box system*, which was a good physical description of the way in which the structure resists lateral forces. However, building codes have dropped this terminology, and most wood-frame shearwall buildings are now classified as *bearing wall systems*. The distinction between the shearwall and diaphragm system and other systems is explained in Chap. 3.

Other types of wood building systems, such as glulam arches, and post-frame (or pole) buildings, are beyond the specific scope of this book. It is felt that the designer should first have a firm understanding of the behavior of basic shearwall buildings and the design procedures that are applied to them. With a background of this nature, the designer can acquire from currently available sources (e.g., Refs. 1.2, 1.8, and 1.12) the design techniques for other systems.

The basic bearing wall system can be constructed entirely from wood components. See Figure 1.1. Here the *roof, floors, and walls* use wood framing. The calculations necessary to design these structural elements are illustrated throughout the text in comprehensive examples.

In addition to buildings that use only wood components, other common types of construction make use of wood components in combination with some other type or types of structural material. Perhaps the most common mix of structural materials is in buildings that use *wood roof and floor systems* and *concrete tilt-up* or *masonry (concrete block or brick) shearwalls*. See Figure 1.2. This type of construction is common, especially in one-story commercial and industrial buildings. This construction is economical for small buildings, but its economy improves as the size of the building increases. Trained crews can erect large areas of *panelized* roof systems in short periods of time. See Figure 1.3.

Design procedures for the wood components used in buildings with concrete or masonry walls are also illustrated throughout this book. The connections between wood and concrete or masonry elements are particularly important and are treated in considerable detail.

This book covers the *complete* design of wood-frame *box*-type buildings from the roof level down to the foundation. In a complete building design, *vertical loads and lateral forces* must be considered, and the design procedures for both are covered in detail.

Wind and seismic (earthquake) are the two lateral forces that are normally taken into account in the design of a building. In recent years, design for lateral forces has become a significant portion of the design effort. The reason for this is an increased awareness of the effects of lateral forces. In addition, the building codes have substantially revised the design requirements for both wind and seismic forces. These changes are the result of extensive research in wind engineering and earthquake-resistant design.

1.3 Required and Recommended References

The *eighth* edition of this book was primarily prompted by the publication of the 2018 edition of the *National Design Specification for Wood Construction* (NDS) (Ref. 1.4) as well as by *Minimum Design Loads and associated criteria for Buildings and Other Structures*

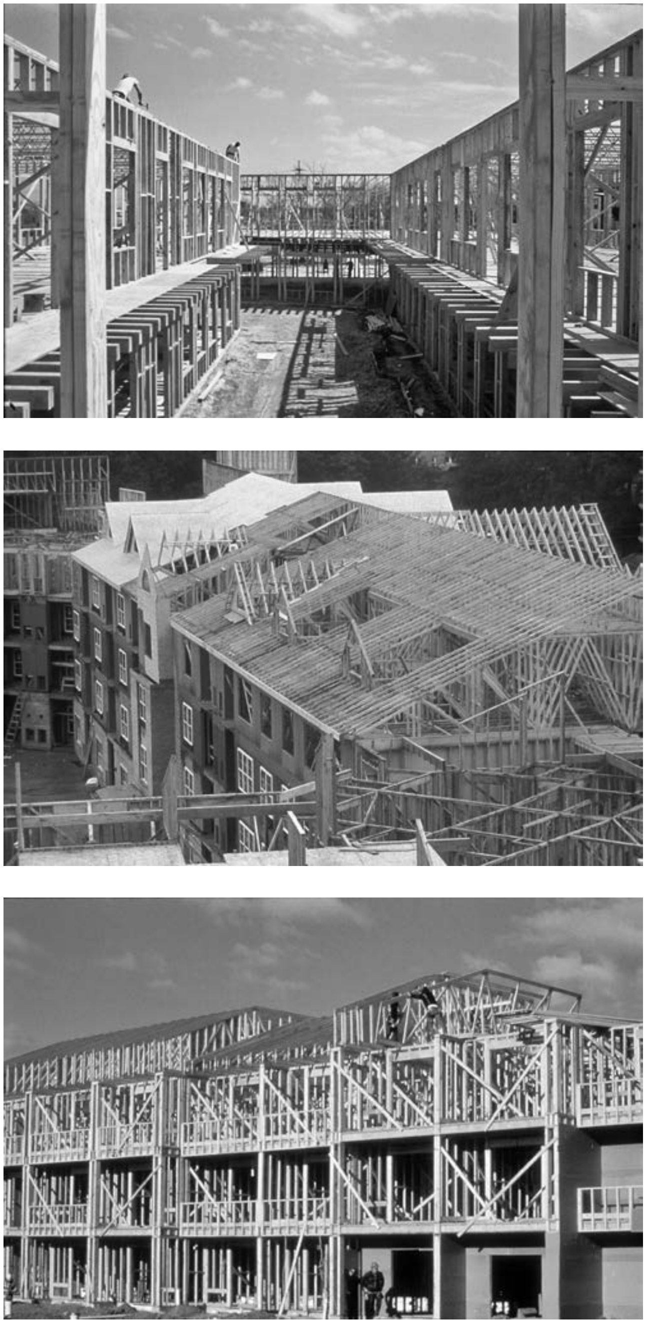

Figure 1.1 Multistory wood-frame buildings. (*Photo courtesy of Southern Pine Council*.)

Figure 1.2a *Foreground:* Office portion of wood-frame construction. *Background:* Warehouse with concrete tilt-up walls and wood roof system. (*Photo courtesy of Mike Hausmann*.)

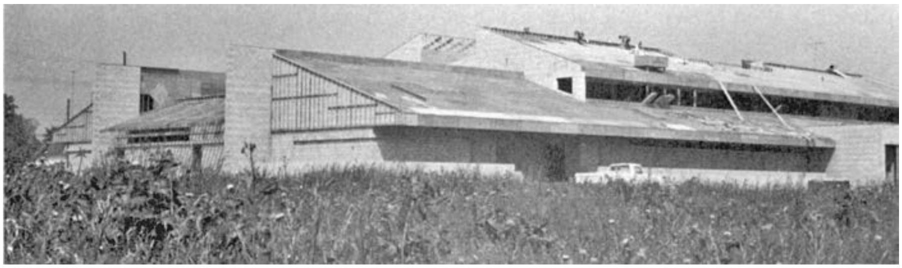

Figure 1.2b Building with reinforced-masonry block walls and a wood roof system with plywood sheathing. (*Photo courtesy of Mark Williams*.)

(ASCE 7-16) (Ref. 1.3) and the 2018 *International Building Code* (IBC) (Ref. 1.9). The 2018 NDS, like previous editions, is in a *dual format*, including both *allowable stress design* (ASD) and *load and resistance factor design* (LRFD) provisions. Editions of the NDS prior to the 2012 edition had been in ASD only.

The NDS is published by the American Wood Council (AWC) and represents the latest structural design recommendations by the wood industry. In addition to basic design provisions for both ASD and LRFD, the 2018 NDS contains chapters specific to

Figure 1.3 Interior of a building with a panelized roof system. (*Photo courtesy of Southern Pine Council*.)

sawn lumber, glued-laminated timber, poles and piles, wood I-joists, structural composite lumber, wood structural panels, cross-laminated timber, mechanical connections, dowel-type fasteners, split ring and shear plate connectors, timber rivets, shearwalls and diaphragms, special loading conditions, and fire design.

The NDS is actually the formal design section of what is a series of interrelated design documents. There are two primary companion documents that support and complete the dual-format *National Design Specification for Wood Construction*. The first companion document is the *NDS Supplement: Design Values for Wood Construction*, which is often referred to simply as the *Supplement* or the *NDS Supplement* as this was the original and for many years the only supplement to the NDS. The NDS Supplement contains all the reference design values for various species groupings of structural lumber and glued-laminated timber. The NDS Supplement is updated at the same time as the NDS, so the current Supplement edition is 2018.

The second companion design document to the NDS is the *Special Design Provisions for Wind and Seismic* (Ref. 1.6), also called the *Wind and Seismic Supplement* or *SDPWS*. The Wind and Seismic Supplement is the newest supplement and is maintained as a separate document due to the unique requirements related to wind- and seismic-resistant design. Included in the SDPWS are reference design values for shearwalls and diaphragms, which comprise the primary lateral-force-resisting system (LFRS) in most wood structures. SDPWS is updated on a different cycle than the NDS, and the current edition is 2015. A new edition will be published in 2021.

The NDS along with both the NDS Supplement and the SDPWS comprises the core of what is needed to design engineered wood structures. Because of the subject matter, the reader must have a copy of the NDS to properly follow this book. Additionally, the numerous tables of member properties, design values, fastener capacities, and unit shears for shearwalls and diaphragms are lengthy. Rather than reproducing these tables in this book, the reader is better served to have copies of both supplements as well. Having a copy of the *NDS*, the *NDS Supplement: Design Values for Wood Construction*, and the *NDS Supplement: Special Design Provisions for Wind and Seismic* is analogous to having a copy of the AISC Steel Manual (Ref. 1.1) in order to be familiar with structural steel design.

Commentary on the NDS provisions is provided in the NDS document. This commentary provides additional guidance and other supporting information for the design provisions included in the NDS.

In addition to the NDS and its two supplements, another associated document is available from the American Wood Council. This document is the *ASD/LRFD Manual for Engineered Wood Construction.* The ASD/LRFD Manual for Engineered Wood Construction was first introduced for ASD in 1999 for the 1997 NDS, and for the first time brought together all necessary information required for the design of wood structures. Prior to this, the designer referred to the NDS for the design of solid sawn lumber and glulam members, as well as the design of many connection details. For the design of other wood components and systems, the designer was required to look elsewhere. The 2018 Manual contains supporting information for both LRFD and ASD, including nonmandatory design information such as span tables, load tables, and fire assemblies. The Manual is organized to parallel the NDS.

All or part of the design recommendations in the NDS are incorporated into the wood design portions of U.S. building codes. With recent codes, this adoption has occurred through adopting by reference (citing as adopted) a particular edition of the

NDS, NDS Supplement, and SDPWS. However, the code change process can take considerable time. This book deals specifically with the design provisions of the 2018 NDS, and the designer should verify local building code acceptance before basing the design of a particular wood structure on these criteria.

This book also concentrates heavily on understanding the loads and forces required in the design of a structure. Emphasis is placed on both gravity loads and lateral forces. Toward this goal, the design loads and forces in this book are taken from the 2018 *International Building Code* (IBC) (Ref. 1.9). The IBC is published by the International Code Council (ICC), and it is highly desirable for the reader to have a copy of the IBC to follow the discussion in this book.

Frequent references are made in this book to the NDS, the NDS Supplements, SDPWS, the ASD/LRFD Manual for Engineered Wood Construction, and the IBC. In addition, a number of cross references are made to discussions or examples in this book that may be directly related to a particular subject. The reader should clearly understand the meaning of the following references:

Examples Reference	Refers to	Where to Look
NDS Section 15.1	Section 15.1 in 2018 NDS	2018 NDS (required reference)
NDS Supplement Table 4A	Table 4A in 2018 NDS Supplement	2018 NDS Supplement (comes with NDS)
SDPWS Supplement Table 4.2A	Table 4.2A in the 2015 Wind and Seismic Supplement	NDS Supplement: Special Design Provisions for Wind and Seismic
IBC Chapter 16	Chapter 16 in 2018 IBC	2018 IBC (recommended reference)
IBC Table 1607.1	Table 1607.1 in 2018 IBC	2018 IBC (recommended reference)
Section 4.15	Section 4.15 of this book	Chapter 4 in this book
Example 9.3	Example 9.3 in this book	Chapter 9 in this book
Figure 5.2	Figure 5.2 in this book	Chapter 5 in this book

Another reference that is often cited in this book is the *Timber Construction Manual* (Ref. 1.2), abbreviated as TCM. This handbook can be considered the basic reference on structural glued-laminated timber. Although it is a useful reference, it is not necessary to have a copy of the TCM to follow this book.

1.4 Building Codes and Design Criteria

Cities and counties across the United States typically adopt a building code to ensure public welfare and safety. Until recently, most local governments used one of the three *regional model codes* as the basic framework for their local building code. The three regional model codes were the

1. *Uniform Building Code* (Ref. 1.10)
2. *The BOCA National Building Code* (Ref. 1.7)
3. *Standard Building Code* (Ref. 1.11)

Generally speaking, the *Uniform Building Code* was used in the western portion of the United States, *The BOCA National Building Code* in the north, and the *Standard*

Building Code in the south. The model codes were revised and updated periodically, usually on a 3-year cycle.

While regional code development had been effective, engineering design now transcends local and regional boundaries. The ICC was created in 1994 to develop a single set of comprehensive and coordinated national model construction codes without regional limitations. IBC is one of the products of the ICC. The first edition of the IBC was published in 2000, with newer editions published every three years. Most regions of the United States have adopted all or part of the IBC at either the state or local level.

The ASCE/SEI standard *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (Ref. 1.3) is commonly referred to as ASCE 7-16 or simply ASCE 7. It serves as the basis for some of the loading criteria in the IBC. The IBC directly references ASCE 7, as does this book.

The IBC is used throughout the text to establish the loading criteria for design. The IBC was selected because it is widely used throughout the United States, and because it represents the latest national consensus with respect to load and force criteria for structural design.

Throughout the text, reference is made to the *Code* and the *IBC*. As noted in the previous section, when references of this nature are used, the design criteria are taken from the 2018 edition of the *International Building Code*.

Although the NDS is used in this book as the basis for determining the allowable (for ASD) or strength level (for LRFD) loads for wood members and their connections, note that the IBC also has a chapter that deals with wood design and construction. While the NDS primarily addresses engineered design provisions, the IBC chapter primarily provides requirements for minimum design, construction, and durability.

The designer should be aware that the local building code is the legal authority, and the user should verify acceptance by the local code authority before applying new principles. This is consistent with general practice in structural design, which is to follow an approach that is both rational and conservative. The objective is to produce structures which are economical and safe.

1.5 ASD and LRFD

The ASD format compares *allowable stresses* of a material to calculated *working stresses* resulting from *service loads.* A single *factor of safety* is applied to the *nominal design value* to arrive at the allowable design value. In the LRFD method, *adjusted capacities* (resistance) are compared to the effects of *factored loads*. The factors are developed for both resistance and loads such that uncertainty and consequences of failure are explicitly recognized.

Basic behavioral equations form the basis for both ASD and LRFD provisions. Therefore, the basic behavior of wood is presented in this text first, followed by ASD and LRFD provisions. The reader should be careful when referencing any equations or examples and confirm that the correct format, whether ASD or LRFD, is being reviewed.

All examples in the text are located in shaded boxes. Where examples are specifically using ASD, the example title includes “using ASD” and where using LFRD, the example title includes “using LFRD”.

1.6 Organization of the Text

The text has been organized to present the complete design of a wood-frame building in an orderly manner. The subjects covered are presented roughly in the order that they would be encountered in the design of a building.

In a building design, the first items that need to be determined are the design loads. The Code requirements for vertical loads and lateral forces are reviewed in Chap. 2, and the distribution of these in a building with wood framing is described in Chap. 3.

Following the distribution of loads and forces, attention is turned to the design of wood elements. As noted previously, there are basically two systems that must be designed, one for *vertical loads* and one for *lateral forces*.

The vertical-load-carrying system is considered first. In a wood-frame building, this system is basically composed of beams and columns. Chapters 4 and 5 cover the characteristics and design properties of these wood members. Chapter 6 then outlines the design procedures for beams, and Chap. 7 treats the design methods for columns and members subjected to combined axial and bending loads.

As one might expect, some parts of the vertical-load-carrying system are also part of the lateral-force-resisting system (LFRS). The sheathing for wood roof and floor systems is one such element. The sheathing distributes the vertical loads to the supporting members, and it also serves as the *skin* or *web* of the diaphragm for resisting lateral forces. Chapter 8 introduces the grades and properties of wood-structural panels and essentially serves as a transition from the vertical-load- to the lateral-force-resisting system. Chapters 9 and 10 deal specifically with the LFRS. In the typical bearing wall type of buildings covered in this text, the LFRS is made up of a diaphragm that spans horizontally between vertical shear-resisting elements known as shearwalls.

After the design of the main elements in the vertical-load- and lateral-force-resisting systems, attention is turned to the design of the connections. The importance of proper connection design cannot be overstated, and design procedures for various types of wood connections are outlined in Chaps. 11 through 14.

Chapter 15 describes the anchorage requirements between horizontal and vertical diaphragms. Basically, anchorage ensures that the horizontal and vertical elements in the building are adequately tied together.

Chapter 16 addresses building code requirements for seismically irregular structures. Chapter 16 also expands the coverage of overturning for shearwalls.

1.7 Structural Calculations

Structural design is at least as much an *art* as it is a *science*. This book introduces a number of basic structural design principles. These are demonstrated through a large number of practical numerical examples and sample calculations. These should help the reader understand the technical side of the problem, but the application of these tools in the design of wood structures is an art that is developed with experience.

Equation-solving software or *spreadsheet* computer programs can be used to create a *template* that can easily generate the solutions of many wood design equations. Using the concept of a template, the design equations need to be entered only once. Then they can be used, time after time, to solve similar problems by changing certain variables.

Equation-solving software and spreadsheet applications relieve the user of many of the tedious programming tasks associated with writing dedicated software. Dedicated computer programs certainly have their place in wood design, just as they do in other areas of structural design. However, equation-solving software and spreadsheets have leveled the playing field considerably. Templates can be simple, or they can be extremely sophisticated. Regardless of programming experience, it should be understood that a simple template can make the solution of a set of bolt equations easier than looking up a design value in a table.

It is highly recommended that the reader become familiar with one of the popular equation-solving or spreadsheet application programs. It is further recommended that a number of the sample problems be solved using such applications. With a little practice, it is possible to create templates which will solve problems that are repetitive and tedious on a hand-held calculator.

Although the power and convenience of equation-solving and spreadsheet applications should not be overlooked, all the numerical problems and design examples in this book are shown as complete hand solutions.

With this in mind, an expression for a calculation is first given in general terms (i.e., a formula is first stated), then the numerical values are substituted in the expression, and finally the result of the calculation is given. In this way, the designer should be able to readily follow the sample calculation.

Note that the conversion from pounds (lb) to kips (k) is often made without a formal notation. This is common practice and should be of no particular concern to the reader. For example, the calculations below illustrate the adjusted axial load capacity of a tension member:

$$
\begin{aligned}
T &= F_t'A \\
&= (1200 \text{ lb/in.}^2)(20 \text{ in.}^2) \\
&= 24 \text{ k}
\end{aligned}
$$

where T = tensile force
F_t' = adjusted tensile design value
A = cross-sectional area

The following illustrates the conversion for the above calculations, which is normally done mentally:

$$
\begin{aligned}
T &= F_t'A \\
&= (1200 \text{ lb/in.}^2)(20 \text{ in.}^2) \\
&= (24{,}000 \text{ lb})\left(\frac{1\text{k}}{1000\text{lb}}\right) \\
&= 24 \text{ k}
\end{aligned}
$$

The appropriate number of significant figures used in calculations should be considered by the designer. When structural calculations are done on a calculator or computer, there is a tendency to present the result with too many significant figures. Variations in loading and material properties make the use of a large number of significant figures inappropriate. A false degree of *accuracy* is implied when the stress in a wood member is recorded in design calculations with an excessive number of significant figures.

As an example, consider the bending stress in a wood beam. If the calculated stress as shown on the calculator is 1278.356 … psi, it is reasonable to report 1280 psi in the design calculations. Rather than representing sloppy work, the latter figure is more realistic in presenting the degree of accuracy of the problem.

Although the calculations for problems in this text are performed on a computer or calculator, intermediate and final results are generally presented with three or four significant figures.

An attempt has been made to use a consistent set of symbols and abbreviations throughout the text. Comprehensive lists of symbols and abbreviations, and their definitions, follow the Contents. A number of the symbols and abbreviations are unique to this book, but where possible they are in agreement with those accepted in the industry. The NDS uses a comprehensive notation system for many of the factors used in the design calculations for wood structures. This notation system is commonly known as the *equation format* for wood design and is introduced in Chap. 4.

The units of measure used in the text are the U.S. Customary System units. The abbreviations for these units are also summarized after the Contents. Factors for converting to SI units are included in App. C.

1.8 Detailing Conventions

With the large number of examples included in this text, the sketches are necessarily limited in detail. For example, a number of the building plans are shown without doors or windows. However, each sketch is designed to illustrate certain structural design points, and the lack of full details should not detract from the example.

One common practice in drawing wood structural members is to place an × in the cross section of a *continuous* wood member. A *noncontinuous* wood member is shown with a single diagonal line in the cross section. See Figure 1.4.

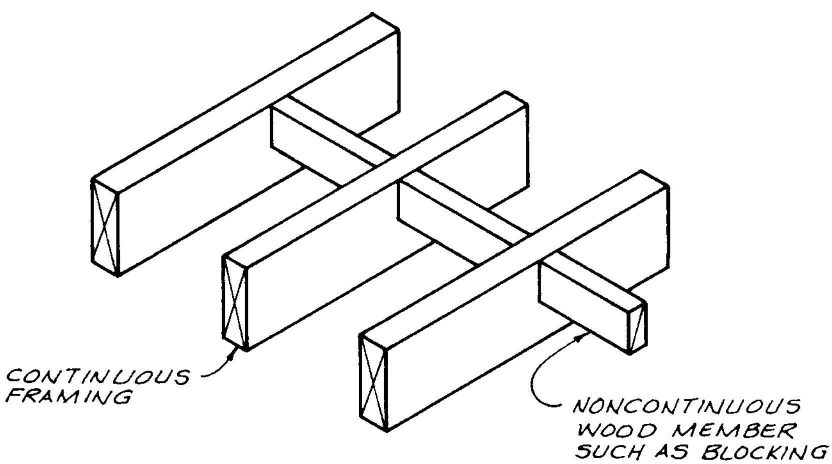

Figure 1.4 Typical timber drafting conventions.

1.9 Fire-Resistive Requirements

Building codes place restrictions on the materials of construction based on the occupancy (i.e., what the building will house), area, height, number of occupants, and a number of other factors. The choice of materials affects not only the initial cost of a building but the recurring cost of fire insurance premiums as well.

The fire-resistive requirements are important to the building designer. This topic can be a complete subject in itself and is beyond the scope of this book. However, several points that affect the design of wood buildings are mentioned here to alert the designer.

Wood (unlike steel and concrete) is a combustible material, and certain *types of construction* (defined by the Code) do not permit the use of combustible materials. There are arguments for and against this type of restriction, but these limitations do exist.

Generally speaking, the *unrestricted use* of wood is allowed in buildings of limited floor area. In addition, the height of these buildings without automatic fire sprinklers is limited to one, two, or three stories, depending upon the occupancy.

Wood is also used in another type of construction known as *heavy timber*. Experience and fire endurance tests have shown that the tendency of a wood member to ignite in a fire is affected by its cross-sectional dimensions. In a fire, large-size wood members form a protective coating of char which insulates the inner portion of the member. Thus, large wood members may continue to support a load in a fire long after an unprotected steel member has collapsed because of the elevated temperature. This is one of the arguments used against the restrictions placed on "combustible" building materials. (Note that properly protected steel members can perform adequately in a fire.)

The minimum cross-sectional dimensions required to qualify for the heavy timber fire rating are set forth in building codes. As an example, the IBC states that the minimum nominal cross-sectional dimension for a solid sawn wood column is 8 in. Different minimum nominal dimensions apply to different types of wood members, and the Code should be consulted for these values. Limits on maximum allowable floor areas are much larger for wood buildings with heavy timber members, compared with buildings without wood members of sufficient size to qualify as heavy timber.

For additional information on fire design of wood members, the reader is referred to NDS in Chap. 16.

1.10 Industry Organizations

A number of organizations are actively involved in promoting the proper design and use of wood and related products. These include the model building code groups as well as a number of industry-related organizations. The names and addresses of some of these organizations are listed after the Contents. Others are included in the list of references at the end of each chapter.

1.11 References

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1.3 American Society of Civil Engineers (ASCE). 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE 7-16*, ASCE, New York, NY.

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1.5 American Wood Council (AWC). 2018. *ASD/LRFD Manual for Engineered Wood Construction*, 2018 ed., AWC, Leesburg, VA.

1.6 American Wood Council (AWC). 2015. *Special Design Provisions for Wind and Seismic*, ANSI/AWC SDPWS-2015, AWC, Leesburg, VA.

1.7 Building Officials and Code Administrators International, Inc. (BOCA). 1999. *The BOCA National Building Code/1999*, 13th ed., BOCA, Country Club Hills, IL.

1.8 FP Innovations, American Wood Council, U.S. Forest Products Laboratory, APA, and Woodworks, *Cross-Laminated Timber Handbook*, 2012 ed., AWC, Leesburg, VA.

1.9 International Code Council (ICC). 2018. *International Building Code*, 2018 ed., ICC, Washington DC.

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1.12 Walker, J.N., and Woeste, F.E. (eds.) 1992. *Post-Frame Building Design Manual*, ASAE—The Society for Engineering in Agricultural, Food, and Biological Systems, St. Joseph, MI.

CHAPTER 2

Design Loads

2.1 Introduction

Calculation of design loads for buildings is covered in this chapter and Chap. 3. This chapter introduces design loads, load combinations, and serviceability criteria for design of wood structures. Chapter 3 is concerned with distribution of these loads throughout the structure.

The types of loads that the designer is asked to consider are defined in American Society of Civil Engineers (ASCE) Standard 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (Ref. 2.5), and the 2018 edition of the *International Building Code* (IBC, Ref. 2.9). Some of these load types act vertically, some act laterally (horizontally), and some include both vertical and horizontal components. The important differentiation for most wood structures is that the vertical portion of the load (acting either up or down) is generally carried by systems of joists, beams, and posts or walls, while loads acting laterally are carried primarily by sheathed shearwalls. For this reason, this text discusses the two primary groups of loads: (1) vertical loads—loads acting on the vertical-load-carrying system and (2) lateral loads—loads acting on the lateral-load-resisting system.

Certain members may function only as vertical-load-carrying members or only as lateral-load-carrying members, while a few members may be subject to a combination of vertical and lateral loads. For example, a member may function as a beam when subjected to vertical loads and as an axial tension or compression member under lateral loads (or vice versa).

Gravity loads offer a natural starting point, as little introduction is required. "Weight" is something with which most people are familiar, and the design for vertical loads is often accomplished first. The reason for starting here is twofold. First, gravity loading is an ever-present load, and quite naturally it has been the basic, traditional design concern. Second, in the case of lateral seismic loads, it is necessary to know the magnitude of the vertical loads before the design seismic loads can be estimated.

The terms load and force are often used interchangeably. Both are used to refer to a vector quantity with U.S. Customary System units of pounds (lb) or kips (k). In keeping with use in ASCE 7, this text primarily uses the term *loads* to indicate vertical, lateral, and other external environmental effects being applied to the structure. The term *forces* will primarily refer to the demand on a member as a result of applied loads. Some variation in use may occur within the text; such variation in use is not intended to affect the meaning.

Also note that the design of structural framing members usually follows the reverse order in which they are constructed in the field. That is, design starts with the lightest framing member on the top level and proceeds downward, and construction starts at the bottom with the largest members and proceeds upward.

Before starting discussion of each load type, one last item bears mentioning. An engineered design approach involves calculating a demand due to loads, and comparing the demand with the capacity of the member or element under consideration. An alternative approach to construction of wood light-frame buildings and some other systems is available in the *International Building Code* (IBC, Ref. 2.9) and the *International Residential Code* (IRC, Ref. 2.10). Other standards may be applicable in high-wind regions. This approach is referred to as *prescriptive construction* because the construction requirements are directly prescribed in code provisions or tables, rather than using calculations of demand and capacity. These provisions permit construction of a limited number of small building types, with the primary intent being one and two family dwellings. Gravity design uses tabulated joist and rafter tables that have been precalculated. Requirements for wind and seismic loads have their basis in historic building practices; minimum lengths of bracing walls and load path connections are prescriptively specified. The wind and seismic bracing provisions will generally result in different construction than would be required for an engineered design. Detailed discussion of prescriptive construction is beyond the scope of this text. The reader is referred to the IBC and IRC and their commentaries for more information on this alternate design approach.

Design loads are the subject of both the IBC and ASCE 7. It is suggested that the reader accompany the remaining portions of this chapter with a review of ASCE 7 and Chap. 16 of the IBC. The load information provided in the IBC is intended to allow the determination of general loading parameters (such as basic wind speed and Seismic Design Category [SDC]) while not necessarily including full details of design loads. This information is placed in the building code to provide easy access to both designers and building officials. It is intended that the designer start with information provided in the IBC, and proceed to more specific information in ASCE 7 for design. There are a few instances, however, when the IBC description of load is more specific than or different from ASCE 7. When conflicts occur between the IBC and referenced standards including ASCE 7, IBC Sec. 102.4.1 requires that the provisions of the IBC apply.

The loadings specified in ASCE 7 and the IBC represent minimum criteria; if the designer has knowledge that the actual loads will exceed the specified minimum loads, the higher values must be used for design. There are also instances where an owner could have interest in higher building performance than will result from minimum code requirements. This is a topic of current discussion in the engineering community around higher than life-safety performance under earthquake loading, but could be applicable for all loading types. In addition, it is required that the structure be designed for loading that can be reasonably anticipated for a given occupancy and structure configuration. ASCE 7 and its commentary provide information on some loads and load combinations that are not addressed in the IBC.

As noted in Chap. 1, this text addresses the national model building code (IBC) and the national consensus standard (ASCE 7). When a design is submitted to a building department, the design must conform to the building code as adopted by that jurisdiction. This will often be either the IBC, or the IBC with further amendments adopted at the state or local level. It is necessary to find out from the building department what code and amendments are adopted.

When using the ASCE 7 standard, the reader should be aware that the earlier printings may have been updated with both errata and supplements. The reader is encouraged to obtain a printing of ASCE 7 that includes these items, or to download the errata and supplements from the ASCE website.

2.2 Dead Loads

The notation D is used to denote dead loads. Dead loads are addressed in ASCE 7 Sec. 3.1 and IBC Sec. 1606. Included in dead loads are the weights of all materials permanently attached to the structure. In the case of a wood roof or wood floor system, this would include the weight of the roof or floor covering, sheathing, framing, insulation, ceiling (if any), and any other permanent materials such as piping, automatic fire sprinkler, and ducts.

Another dead load that must be included is the weight of fixed equipment. Mechanical or air-conditioning equipment on a roof is one example that is easily overlooked. Often this type of load is supported by a built-up member (two or three joists side by side) as shown in Figure 2.1. The alternative is to design special larger and deeper beams to carry these isolated equipment loads.

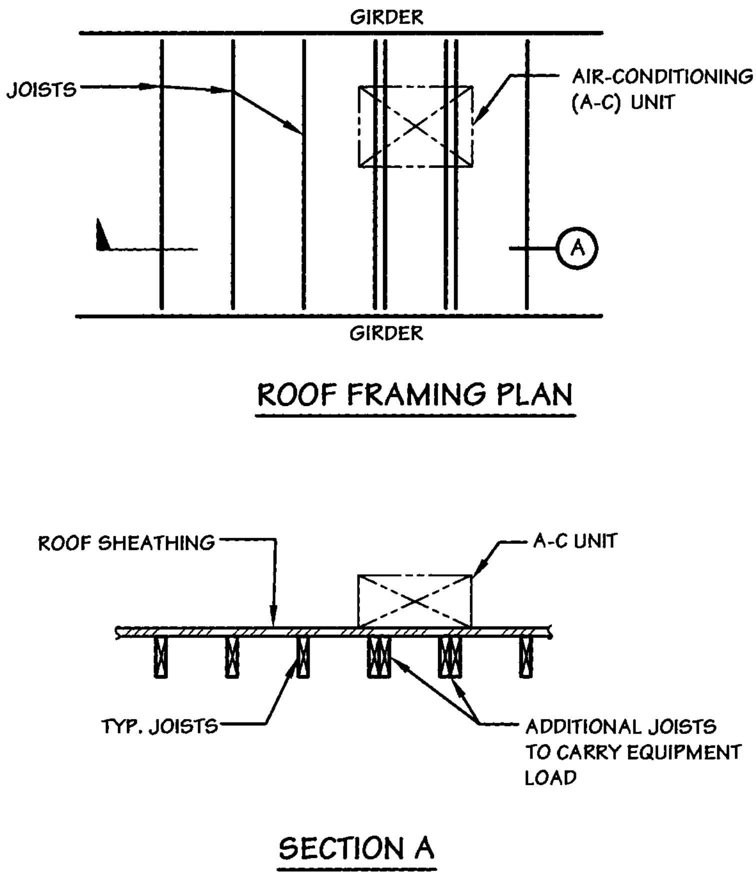

Figure 2.1 Support of equipment loads by additional framing.

The magnitudes of dead loads for various construction materials can be found in a number of references. A fairly complete list of weights is given in App. B, and additional tables are given in Refs. 2.1 and 2.5.

Because most building dead loads are estimated as uniform loads in terms of pounds per square foot (psf), it is often convenient to convert the weights of framing members to these units. For example, if the weight per lineal foot of a wood framing member is known, and if the center-to-center spacing of parallel members is also known, the dead load in psf can easily be determined by dividing the weight per lineal foot by the center-to-center spacing. For example, if 2×12 joists weighing 4.3 lb/ft are spaced at 16 in. on center (o.c.), the equivalent uniform load is $4.3 \text{ lb/ft} \div 1.33 \text{ ft} = 3.2$ psf. A table showing these equivalent uniform loads for typical framing sizes and spacings is given in App. A.

It should be pointed out that in a wood structure, the dead load of the *framing members* usually represents a fairly minor portion of the total design load. For this reason, a small error in estimating the weights of framing members (either lighter or heavier) typically has a negligible effect on the final member choice. *Slightly* conservative (larger) estimates are preferred for design.

The estimation of the dead load of a structure requires some knowledge of the methods and materials of construction. A "feel" for what the unit dead loads of a wood-frame structure should total is readily developed after exposure to several buildings of this type. The dead load of a typical wood floor or roof system usually ranges between 7 and 20 psf, depending on the materials of construction, span lengths, and whether a ceiling is suspended below the floor or roof. For wood wall systems, values might range between 4 and 20 psf, depending on stud size and spacing and the type of wall sheathing used (for example, $^3/_8$-in. plywood weighs approximately 1 psf whereas $^7/_8$-in. stucco weighs 10 psf of wall surface area). Typical load calculations provide a summary of the makeup of the structure. See Example 2.1.

EXAMPLE 2.1 Sample Dead Load D Calculation Summary

Roof Dead Loads

Roofing (5-ply with gravel)	=	6.5 psf
Reroofing	=	2.5
$^1/_2$-in. plywood (3 psf $\times$ $^1/_2$ in.)	=	1.5
Framing (estimate 2×12 at 16 in. o.c.)	=	2.9
Insulation	=	0.5
Suspended ceiling (acoustical tile)	=	1.0
Roof dead load D	=	14.9 psf
Say Roof D	=	15.0 psf

Floor Dead Loads

Floor covering (lightweight concrete $1^1/_2$ in. at 100 lb/ft^3)	=	12.5
$1^1/_8$-in. plywood (3 psf $\times$ $1^1/_8$ in.)	=	3.4
Framing (estimate 4×12 at 4 ft-0 in. o.c.)	=	2.5
Ceiling supports (2×4 at 24 in. o.c.)	=	0.6
Ceiling ($^1/_2$-in. drywall, 5 psf $\times$ $^1/_2$ in.)	=	2.5
Floor dead load D	=	21.5 psf
Say Floor D	=	22.0 psf

The dead load of a wood structure that differs substantially from the typical ranges mentioned above should be examined carefully to ensure that the various individual dead load (D) components are in fact correct. It pays in the long run to stand back several times during the design process and ask, "Does this figure seem reasonable compared with typical values for other similar structures?"

In the summary of roof dead loads in Example 2.1, the load titled *reroofing* is sometimes included to account for the weight of roofing that may be added at some future time. Subject to the approval of the local building official, new roofing materials may sometimes be applied without the removal of the old roof covering. Depending on the materials (e.g., built-up, asphalt shingle, wood shingle), one or two overlays may be permitted.

Before moving on to another type of loading, the concept of the *tributary area* of a member should be explained. The area that is assumed to load a given member is known as the tributary area A_T. For a beam or girder, this area can be calculated by multiplying the *tributary width* times the span of the member. See Example 2.2. The tributary width is generally measured from midway between members on one side of the member under consideration to midway between members on the other side. For members spaced a uniform distance apart, the tributary width is equivalent to the spacing between members. Since tributary areas for adjacent members do not overlap, all distributed loads are assumed to be supported by the nearest structural member. When the load to a member is uniformly distributed, the load per foot can readily be determined by taking the unit load in psf times the tributary width (lb/ft^2 × ft = lb/ft). The concept of tributary area will play an important role in the calculation of many types of loads.

Note, however, that a tributary area approach should only be used when the loading is uniform. Where loading varies, a more detailed calculation is required.

For the purpose of determining required live load, an area larger than the tributary area A_T is assumed to influence the structural performance of a member. This area is known as the *influence area* and is calculated in ASCE 7 and the IBC as the *live load element factor* K_{LL} times A_T. K_{LL}, found in ASCE 7 Table 4.7-1 and IBC Table 1607.11.1, varies between 4 and 1. The influence area $K_{LL} \times A_T$ typically includes the full area of all members that are supported by the member under consideration. As discussed previously, the tributary area approach assumes that each load on a structure is supported entirely by the nearest structural member. In contrast, the influence area recognizes that the total load experienced by

EXAMPLE 2.2 Tributary Areas

In many cases, a uniform spacing of members is used throughout the framing plan. This example is designed to illustrate the *concepts* of tributary area rather than typical framing layouts. See Figure 2.2.

Tributary Area Calculations

	A_T = tributary width × span	$A_T \times K_{LL}$
Joist J1	$A_T = 2 \times 12 = 24$ ft^2	$24 \times 2 = 48$ ft^2
Joist J2	$A_T = 2 \times 14 = 28$ ft^2	$28 \times 2 = 56$ ft^2
Girder G1	$A_T = (^{12}/_2 + {}^{14}/_2)20 = 260$ ft^2	$260 \times 2 = 520$ ft^2
Girder G2	$A_T = (^{12}/_2 + {}^{14}/_2)24 = 312$ ft^2	$312 \times 2 = 624$ ft^2
Interior column C1	$A_T = (^{12}/_2 + {}^{14}/_2)(^{20}/_2 + {}^{24}/_2) = 286$ ft^2	$286 \times 4 = 1144$ ft^2
Exterior column C2	$A_T = (^{12}/_2)(^{20}/_2 + {}^{24}/_2) = 132$ ft^2	$132 \times 4 = 528$ ft^2
Corner column C3	$A_T = (^{14}/_2)(^{20}/_2) = 70$ ft^2	$70 \times 2 = 140$ ft^2

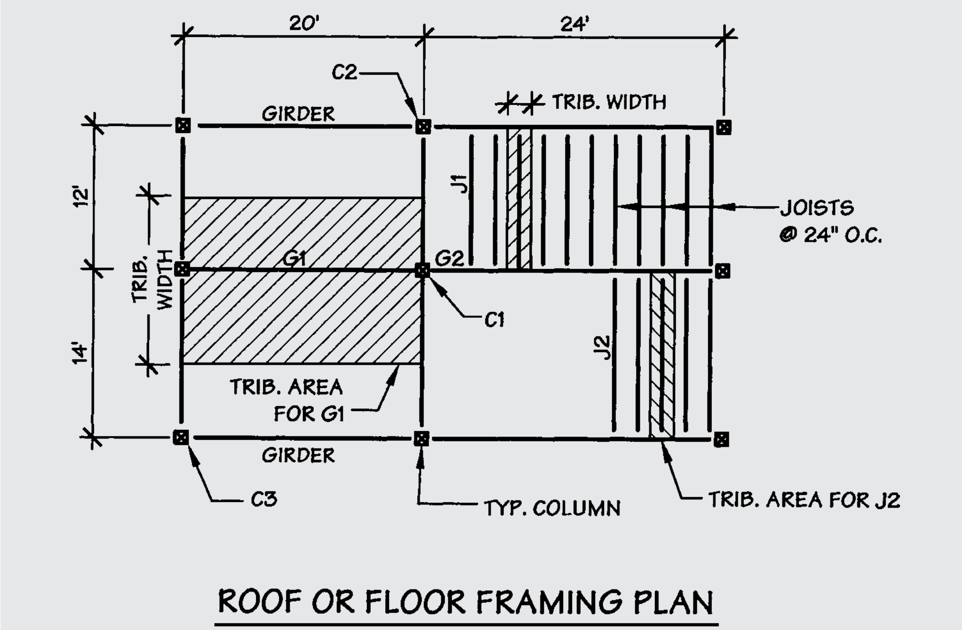

Figure 2.2 Concepts of tributary area.

a structural member may be influenced by loads applied outside the tributary area of the member. For example, any load applied between two beams is recognized to influence both beams, even though the load is located within the tributary area of just one of the beams. For most columns the influence area is $4 \times A_T$ and for most beams the influence area is $2 \times A_T$. Thus, the area represented by influence area will overlap for adjacent beam members or column members. Further discussion can be found in the commentary to ASCE 7 Sec. 4.7.

2.3 Live Loads

The term L_r is used to denote *roof live loads*. The symbol L is used for live loads other than roof. Included in *live loads* are loads associated with *use* or *occupancy* of a structure. Roof live loads are generally associated with maintenance of the roof. While dead loads are applied permanently, live loads tend to fluctuate with time. Typically included are people, furniture, contents, and so on. ASCE 7 and IBC specify the minimum roof live loads L_r and minimum floor live loads L that must be used in the design of a structure. For example, ASCE 7 Table 4.3-1 and IBC Table 1607.1 specify unit floor live loads in psf for the design of floors, and unit roof live loads in psf for the design of roofs. Tabulated floor and roof live loads are designated with the notation L_0, indicating that it is a tabulated, unreduced live load. Permitted live load reductions will be discussed shortly. Note that this book uses the italicized terms L, L_1, and L_2 to denote span. The variable L denoting a span will always be shown in italics, while L_r denoting live loads will be shown in standard text.

The minimum live loads in ASCE 7 and the IBC are, with some exceptions, intended to address only the use of the structure in its final and occupied configuration. Construction means and methods, including loading and bracing during construction, are generally not taken into account in the design of the building. This is because these loads can typically only be controlled by the contractor, not the building designer. In wood-frame structures, the construction loading can include stockpiling of construction materials on the partially completed structure. It is incumbent on the contractor to ensure that such loading does not exceed the capacity of the structural members.

ASCE 7 and the IBC allow for reduction of floor and roof live loads below the tabulated loads. The notation used for the tabulated unreduced live load L_0, when reduced becomes reduced floor live load L, and reduced roof live load L_r. When determining allowable reductions on roof live loads, the tributary area concept is used, while for floor live loads the influence area is used. The concept that the area should be considered in determining the magnitude of the unit uniform live load, not just the total load, is as follows:

> If a member has a small area contributing live load, it is likely that a fairly high unit live load will be imposed over that relatively small area. On the other hand, as the area contributing live load becomes large, it is less likely that this large area will be uniformly loaded by the same high unit load considered in the design of a member with small area.

Therefore, the consideration of the area contributing load (tributary area A_T or influence area $K_{LL} \times A_T$) in determining the unit live load has to do with the probability that high unit loads are likely to occur over small areas, but that high unit loads will probably not occur over large areas.

It should be pointed out that no reduction is permitted where live loads exceed 100 psf, in areas of public assembly, or in parking garages. Reduction is not allowed because an added measure of safety is desired in these critical structures. In warehouses with high storage loads and in areas of public assembly (especially in emergency situations), it is possible for high unit loads to be distributed over large plan areas. However, for the majority of wood-frame structures, reductions in live loads will be allowed.

Floor Live Loads

As noted earlier, minimum floor unit live loads are specified in ASCE 7 Table 4.3-1 and IBC Table 1607.1. These loads are based on the occupancy or use of the building. Typical occupancy or use floor live loads range from a minimum of 30 psf for residential structures to values as high as 250 psf for heavy storage facilities. These tabulated unit live loads, denoted L_0, are for members supporting small influence areas. A small influence area is defined as less than 400 ft^2. From previous discussion of influence areas, it will be remembered that the magnitude of the unit live load can be reduced as the size of the influence area increases. For members with an influence area of $K_{LL} \times A_T \geq 400$ ft^2, the reduced live load L is determined as follows:

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} \times A_T}}\right)$$

where L_0 = unreduced floor live load specified in IBC Table 1607.1 or ASCE 7 Table 4.3-1. The reduced live load L is not permitted to be less than 0.5 L_0 for members supporting loads from only one floor of a structure, nor less than 0.4 L_0 for members supporting loads from two or more floors.

The calculation of reduced floor live loads is illustrated in Example 2.3.

EXAMPLE 2.3 Reduction of Floor Live Loads Using ASD

Determine the total axial force required for the design of the interior column in the floor- framing plan shown in Figure 2.3. The structure is an apartment building with a floor dead load D of 10 psf and, from ASCE 7 Table 4.3-1, a tabulated floor live load L_0 of 40 psf. Assume that roof loads are not part of this problem and the load is received from one level.

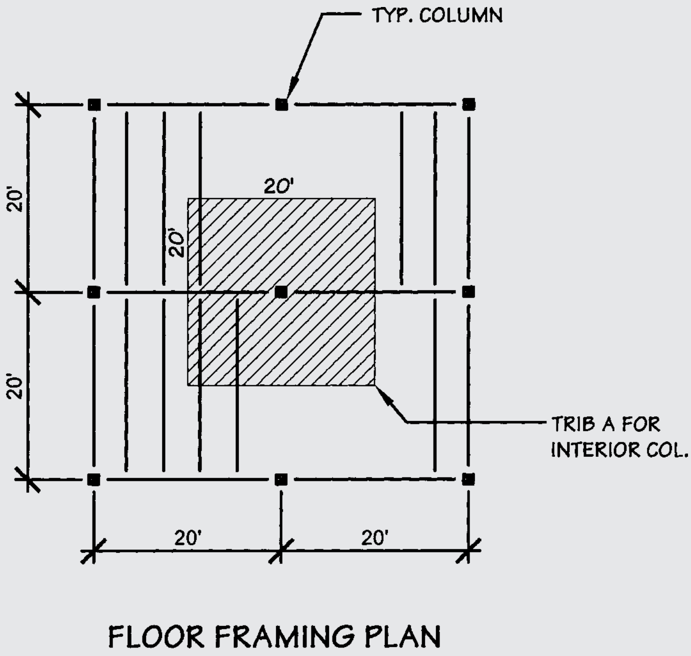

FIGURE 2.3 Interior column tributary area.

Floor Live Load

$$\textit{Tributary } A = A_T = 20 \times 20 = 400 \text{ ft}^2$$

$$K_{LL}A_T = 4(400) = 1600 \text{ ft}^2 > 400 \text{ ft}^2$$

∴ floor live load can be reduced

$$L_0 = 40 \text{ psf}$$

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} \cdot A_T}}\right) = 40\left(0.25 + \frac{15}{\sqrt{1600}}\right) = (40)(0.625) = 25 \text{ psf}$$

Check:

$$0.5L_0 = 0.5(40) = 20 \text{ psf} < 25 \text{ psf} \qquad \textit{OK}$$

Use L = 25 psf.

Total Load

$$TL = D + L = 10 + 25 = 35 \text{ psf}$$

$$P = 35 \times 400 = 14{,}000 \text{ lb} = 14.0 \text{ k}$$

In addition to basic floor uniform live loads in psf, ASCE 7 and the IBC provide special alternate concentrated floor loads. The type of live load, uniform or concentrated, which produces the more critical condition in the required load combinations (Sec. 2.18) is to be used in sizing the structure.

Concentrated floor live loads other than vehicle wheel loads can be distributed over an area $2^1/_2$ ft square ($2^1/_2$ ft by $2^1/_2$ ft). Their purpose is to account for miscellaneous nonstationary equipment loads which may occur. Vehicle loads are required to be distributed over an area of 20.25 in.^{2} (4.5 in. by 4.5 in.), which is approximately the contact area between a typical car jack and the supporting floor.

It will be found that the majority of designs will be governed by the uniform live loads. However, both the concentrated loads and the uniform loads should be checked. For certain wood-framing systems, NDS Sec. 15.1 (Ref. 2.3) provides a method of distributing concentrated loads to adjacent parallel framing members.

IBC Sec. 1607.11.2 also provides an alternate method of calculating floor live load reductions based on the tributary area of a member. The formula for calculating the live load reduction is different from the formula for the influence area approach.

In buildings where partitions will likely be erected, use of a uniform partition live load is required in addition to the floor live load per ASCE 7 Sec. 4.3.2 and IBC Sec. 1607.5. The partition live load applies whether or not partitions are shown on the plans. This creates an allowance for addition or reconfiguration of partitions, which occurs frequently as part of tenant improvements of office, retail, and similar spaces. ASCE 7 and the IBC require a minimum live load of 15 psf in addition to floor live load. Where the weight of planned partitions is greater than 15 psf, the actual weight should be used in design. ASCE 7 and the IBC do not require the partition live load where the design floor live load is 80 psf or greater. In past codes this allowance for partitions has been defined as a dead load. Live load reduction does not apply to the partition live load, as both ASCE 7 and the IBC are specific that live load reduction applies only to loads specified in ASCE 7 Table 4.3-1 or IBC Table 1607.1.

Roof Live Loads

ASCE 7 and the IBC specify minimum unit live loads that are to be used in the design of a roof system. The live load on a roof is usually applied for a relatively short period of time during the life of a structure. This fact is normally of no concern in the design of structures other than wood. However, as will be shown in subsequent chapters, the length of time for which a load is applied to a *wood structure* does have an effect on the capacity (resistance).

Roof live loads are specified to account for the miscellaneous loads that may occur on a roof. These include loads that are imposed during the roofing process. Roof live loads that may occur after construction include reroofing operations, air-conditioning and mechanical equipment installation and servicing, and, perhaps, loads caused by fire-fighting equipment. Wind forces and snow loads are not normally classified as live loads, and they are covered separately.

Unit roof live loads are calculated based on the provisions of ASCE 7 Secs. 4.3 and 4.8 and IBC Sec. 1607.13. The standard roof live load for small tributary areas on flat roofs is 20 psf. A reduced roof live load may be determined based on the slope or pitch of the roof and the tributary area of the member being designed. The larger the tributary area, the lower the unit roof live load. As discussed for floor live load reductions, the consideration of tributary area has to do with the reduced probability of high unit loads occurring over large areas. Consideration of roof slope also relates to the probability of loading. On a roof that is relatively flat, fairly high unit live loads are likely to occur. However, on a steeply pitched roof much smaller unit live loads will be probable.

Reduced roof live loads may be calculated for tributary areas A_T greater than 200 ft^2 and for roof slopes F steeper than 4 in./ft, as illustrated in the following formula:

$$L_r = L_0 R_1 R_2 \quad \text{and} \quad 12 \le L_r \le 20 \text{ psf}$$

$$\text{Where } R_1 = \begin{cases} 1 & \text{for } A_T \le 200 \text{ ft}^2 \\ 1.2 - 0.001A_T & \text{for } 200 \text{ ft}^2 < A_T < 600 \text{ ft}^2 \\ 0.6 & \text{for } A_T \ge 600 \text{ ft}^2 \end{cases}$$

$$R_2 = \begin{cases} 1 & \text{for } F \le 4 \\ 1.2 - 0.05F & \text{for } 4 < F < 12 \\ 0.6 & \text{for } F \ge 12 \end{cases}$$

A_T = tributary area supported by structural member, ft^2
F = the number of inches of rise per foot of run for a sloped roof
L_0 = minimum uniform live load per ASCE 7 Table 4.3-1 or IBC Table 1607.1

The above calculations and limits are for what are noted as "ordinary" roofs. Where roofs serve special functions, uniform live loads as high as 100 psf are required.

Example 2.4 illustrates the determination of roof live loads for various structural members based on the tributary area of each member.

EXAMPLE 2.4 Calculation of Roof Live Loads Using ASD

Determine the uniformly distributed roof loads (including dead load and roof live load) for the purlins and girders in the building shown in Figure 2.4. Also determine the total load on column C1. Assume that the roof is flat (except for a minimum slope of $^1/_4$ in./ft for drainage). Roof dead load D = 8 psf.

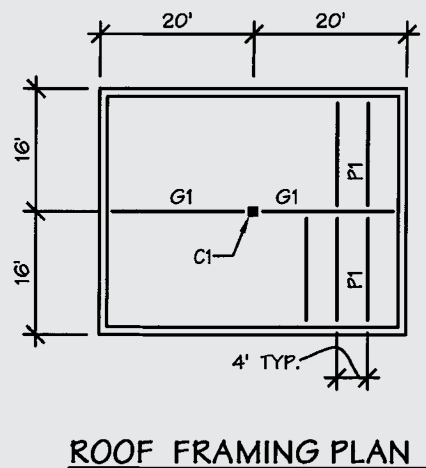

Figure 2.4 Example building plan.

Tributary Areas

Purlin P1: $A_T = 4 \times 16 = 64\ \text{ft}^2$

Girder G1: $A_T = 16 \times 20 = 320\ \text{ft}^2$

Column C1: $A_T = 16 \times 20 = 320\ \text{ft}^2$

Roof Loads

Flat roof:

$$\therefore\ R_2 = 1$$

a. Purlin

$$A_T = 64\ \text{ft}^2 < 200\ \text{ft}^2$$
$$\therefore\ R_1 = 1$$
$$\therefore\ \text{L}_r = 20\ \text{psf}$$
$$w = (\text{D} + \text{L}_r)\ (\text{tributary width}) = [(8 + 20)\ \text{psf}]\ (4\ \text{ft}) = 112\ \text{lb/ft}$$

b. Girder

$$200\ \text{ft}^2 < (A_T = 320\ \text{ft}^2) < 600\ \text{ft}^2$$
$$R_1 = 1.2 - 0.001A_T = 1.2 - 0.001(320) = 0.88$$
$$\text{L}_r = 20R_1R_2 = (20)(0.88)(1) = 17.6\ \text{psf}$$
$$w = [(8 + 17.6)\ \text{psf}]\ (16\ \text{ft}) = 409.6\ \text{lb/ft}$$

c. Column

$$A_T = 320\ \text{ft}^2 \quad \text{same as girder}$$
$$\therefore\ \text{L}_r = 17.6\ \text{psf}$$
$$P = [(8 + 17.6)\ \text{psf}](320\ \text{ft}^2) = 8192\ \text{lb}$$

It should be pointed out that the unit live loads specified in ASCE 7 and the IBC are applied on a horizontal plane. Therefore, roof live loads on a flat roof can be added directly to the roof dead load. In the case of a sloping roof, the dead load would probably be estimated along the sloping roof; the roof live load, however, would be on a horizontal plane. In order to be added together, the roof dead load or live load must be converted to a load along a length consistent with the load to which it is added. Note that both the dead load and the live load are gravity loads, and they both, therefore, are *vertical* (not inclined) vector resultant forces. See Example 2.5.

In certain framing arrangements, unbalanced live loads (or snow loads) can produce a more critical design situation than loads over the entire span. Should this occur, ASCE 7 and the IBC require that unbalanced loads be considered.

EXAMPLE 2.5 Combined $\text{D} + \text{L}_r$ on Sloping Roof Using ASD (Figure 2.5)

The total roof load ($\text{D} + \text{L}_r$) can be obtained either as a distributed load along the roof slope or as a load on a horizontal plane. The lengths L_1 and L_2 on which the loads are applied must be considered.

Equivalent total roof loads ($\text{D} + \text{L}_r$):

Load on horizontal plane:

$$w_{\text{TL}} = w_{\text{D}}\left(\frac{L_1}{L_2}\right) + w_{\text{L}_r}$$

Load along roof slope:

$$w_{\text{TL}} = w_{\text{D}} + w_{\text{L}_r}\left(\frac{L_2}{L_1}\right)$$

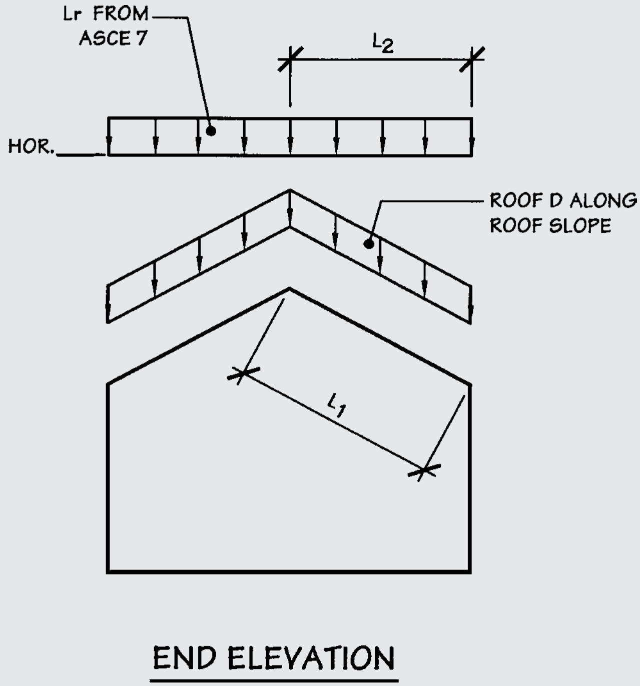

FIGURE 2.5 Combined D + L_r on sloping roof.

Special Live Loads

IBC Sec. 1607 and ASCE 7 Chap. 4 also require design for special loads. Because these loads have to do with the occupancy and use of a structure and tend to fluctuate with time, they are identified as live loads. It should be noted that the direction of these live loads is horizontal in some cases. Examples of special live loads include ceiling vertical live loads and live loads to hand rails and guard rails (which are applied both horizontally and vertically). The notation L is generally used for all live loads other than roof live loads L_r.

2.4 Snow Loads

The notation S is used for snow loads. Snow loads are addressed in ASCE 7 Chap. 7 and IBC Sec. 1608. IBC Sec. 1608 merely directs the user to ASCE 7 Chap. 7 for determination of design snow loads, and reprints ASCE 7 ground snow load maps for the convenience of the IBC user. The discussion in this section will be based on the ASCE 7 provisions.

Snow load is another type of gravity load that primarily affects roof structures. In addition, certain types of floor systems, including balconies and decks may be subjected to snow loads.

The magnitude of snow loads can vary greatly over a relatively small geographic area. As an example of how snow loads can vary, the design snow load in a certain mountainous area of southern California is 100 psf, but approximately 5 mi away at the same elevation, the snow load is only 50 psf. This emphasizes the need to be aware of local conditions. ASCE 7 Figure 7.2-1 provides ground snow loads for many areas of the United States. New in the 2016 edition of ASCE 7 are tabulated ground snow loads for some states, provided in Tables 7.2-1 through 7.2-8. These states are shaded in the Figure 7.2-1 map, and the user is directed to the applicable tabulation of ground snow loads, listed by city or town. Also of note in the Figure 7.2-1 map are areas with gray shading and a designation of "CS." These are areas where site-specific case study of ground snow load is required. This often occurs in mountainous areas where depth of snow and resulting snow load can vary significantly over short distances. Reference to additional snow load information can be found in the footnotes to the ASCE 7 tables. Often the local building department will have established guidance for minimum design snow loads. It is generally a good idea to verify design snow loads with the building department, even beyond CS regions.

Snow loads can be extremely large. For example, a ground snow load of 240 psf is required in an area near Lake Tahoe, California. It should be noted that the specified design snow loads are on a horizontal plane (similar to roof live loads). Unit snow loads (psf), however, are not subject to the tributary area reductions that can be used for roof live loads.

This section will provide an introduction to calculation of snow loads for flat and sloped roofs. For those involved in building design for snow load, there are a number of nuances for snow load calculations. The reader is referred to the full text of ASCE 7 Chap. 7.

Calculation of snow load for flat and sloped roofs uses different formulas. Calculation of snow load for flat roofs uses the formula:

$$S = p_f = 0.7C_e\,C_t\,I_s\,p_g \quad \text{and} \quad p_f \geq p_g I_s \text{ where } p_g \leq 20 \text{ psf}$$
$$p_f \geq 20I_s \text{ where } p_g > 20 \text{ psf}$$

where C_e = exposure factor
C_t = thermal factor
I_s = snow importance factor
p_g = ground snow load (psf)

Each of the expressions in this equation are explained in further detail as follows:

P_g = Ground Snow Load

Ground snow loads are determined in accordance with ASCE 7 Figure 7.2-1 and Tables 7.2-1 through 7.2-8, IBC Figure 1608.2, by the local building department, or by a site-specific study per requirements of ASCE 7. The snow loads on the maps are associated with an annual probability of exceedance of 0.02 (mean recurrence interval of 50 years). Ground snow loads shown on the map for many locations in the western and northwestern regions of the United States are applicable only below specified elevations. Snow loads for higher regions should be determined based on site-specific data and historical records. In the formula given above for design snow load, ground snow load is reduced by a factor of 0.7 to account for the fact that snow accumulation on the ground is greater than at the roof level for most structures. Note that the snow load map in the 2018 IBC corresponds to the map published in ASCE 7-10. This is because the map did not get updated in the code development process leading up to the 2018 Edition of the IBC. IBC Sec. 1608 indicates that either map can be used. In general it is appropriate to use the most recent map (ASCE 7 map) as this represents the most current thinking.

I_s = Snow Importance Factor

The concept behind the snow importance factor is that certain structures should be designed for larger loads than ordinary structures. ASCE 7 lists snow importance factors in Table 1.5-2 as a function of the building's *risk category*. Understanding importance factors requires an explanation of risk categories. Explanations of risk category can be found in two locations: ASCE 7 Sec. 1.5 and Table 1.5-1, and IBC Sec. 1604.5 and Table 1604.5. Unlike other instances where the same information appears in both ASCE 7 and the IBC, there are some differences between risk category descriptions in ASCE 7 and the IBC. The differences do not often impact standard occupancy buildings, so this book uses the ASCE 7 categories. The reader is reminded that the IBC assignments of risk category must be used where conformance to the IBC is required.

In previous editions of ASCE 7 and the IBC, the term *occupancy category* was used in place of *risk category*. The term was revised for the 2010 version of ASCE 7 to better reflect intent. Risk categories vary between I and IV, and give an indication of the relative hazard to life that may be posed, based on the occupants and materials contained in the building as well as the anticipated building use. Risk Category I is identified by IBC Table 1604.5 or ASCE 7 Table 1.5-1 as applicable to buildings and other structures that represent a low risk to human life in the event of a failure. The ASCE 7 commentary identifies barns, storage shelters, gatehouses, and similar structures as typically falling in this risk category. As the risk category increases, the potential hazard increases. Risk Category III includes buildings in which large groups of people congregate, those where occupants may not be able to freely exit in case of emergency due to age, ability, or physical restraint, and buildings housing hazardous substances. This can often include wood-frame school buildings and community centers. Risk category IV includes primarily essential facilities, relied on to be operational following extreme load events. This can include wood-frame fire stations. Buildings not assigned to Risk Categories I, III, or IV are assigned to II. Risk Category II includes the majority of wood-frame buildings.

Going back to ASCE 7 Table 1.5-2, importance factors for snow increase with increasing risk category. This results in higher risk category facilities being designed to support heavier snow loads, therefore reducing likelihood of failure. ASCE 7 snow importance factors are

Risk Category	Importance Factor
I	0.8
II	1.0
III	1.1
IV	1.2

A snow importance factor of 1.0 will be used for the majority of wood-frame buildings, corresponding to a risk category of II. Note that ASCE 7 uses the notation I_s for snow importance factor, and I_e for seismic importance factor. The numerical values of importance factors vary for different load types for a given building. The reader is cautioned to keep careful track of varying I values.

C_e = Snow Exposure Factor

ASCE 7 lists snow exposure factors in Table 7.3-1. The snow exposure factor varies as a function of the *surface roughness category* and exposure of the roof. This is because an exposed roof on an exposed building is likely to have snow blow off, whereas a sheltered roof on a sheltered building is likely to have snow accumulation.

In order to use the exposure surface roughness definitions, *surface roughness categories* must first be determined. These are designated as B, C, and D, and are defined in ASCE 7 Sec. 26.7.2. Surface Roughness B has numerous closely spaced obstructions (i.e., urban and suburban areas, or wooded areas). Surface Roughness C has open terrain with scattered obstructions with heights generally less than 30 ft (i.e., flat open country, grasslands). Surface Roughness D has flat unobstructed areas (i.e., mudflats, salt flats, and unbroken ice).

The fourth terrain category is "above the tree line in windswept mountain areas," and the fifth applies "in Alaska, in areas where trees do not exist within a 2-mi radius of the site." Further explanation of these is not required.

In addition, the exposure of the roof must be chosen from one of the three possible categories: fully exposed, partially exposed, and sheltered. The ASCE 7 commentary indicates that the category is intended to be separately selected for each roof on a building that has more than one roof. Footnote "a" of Table 7.3-1 provides explanation of these exposures. Fully exposed occurs when roofs are exposed on all sides without shelter from higher terrain, mechanical equipment, or conifer trees with leaves in the wintertime. A sheltered roof is defined as a roof located tight in among conifers that qualify as obstructions. Partially exposed roofs are those that do not qualify as fully exposed or sheltered.

As a function of the surface roughness category and exposure of the roof, the snow exposure factor can range between 0.7 and 1.2.

C_t = Thermal Factor

As the name implies, the thermal factor varies based on the thermal condition of the roof of a structure. Thermal factors are provided in ASCE 7 Table 7.3-2. For unheated structures, or for buildings with well-ventilated roofs that have high thermal resistance (*R*-values) and will remain relatively cold during winter months, thermal factors greater than unity are specified since heat transfer from inside the structure will not melt much of the snow on the roof. For continuously heated greenhouses with roofs that have low thermal resistance (*R*-values), a thermal factor of $C_t = 0.85$ is specified since heat transfer from within the structure will tend to melt substantial amounts of snow on the roof. All other structures are assigned a thermal factor of $C_t = 1.0$.

Calculation of snow load for sloped roofs uses the formula:

$$\mathrm{S} = p_s = C_s\, p_f$$

where C_s = slope factor
p_f = flat roof snow load (psf)

C_s = Roof Slope Factor

The roof slope factor is specified in ASCE 7 Sec. 7.4 and provides reduced snow loads based on roof slope, type of roof surface, and thermal condition of the roof. The roof slope factor is intended to address the likelihood of snow sliding to the ground from a sloped roof. Roof surfaces are classified as either "unobstructed slippery surfaces" (e.g., metal, slate, glass, or membranes with smooth surfaces) that facilitate snow sliding from the roof, or as "all other surfaces" (including asphalt shingles, wood shakes or shingles, and membranes with rough surfaces). The thermal condition of a roof is categorized as either "warm" (roofs with $C_t \leq 1.0$) or "cold" (roofs with $C_t > 1.0$) per ASCE 7 Table 7.3-2. As provided in ASCE 7 Figure 7.4-1, for each category of roof, the roof slope factor varies linearly between unity and zero for a specific range of roof slopes. For example, warm roofs that are not slippery or unobstructed (a typical condition for many wood-frame structures) are assigned the following C_s values:

$C_s = 1$ for roof slopes less than 30 degrees (slopes of approximately 7 in./ft or less)

$C_s = 0$ for roof slopes greater than 70 degrees

C_s varies linearly between 1 and 0 for roof slopes between 30 degrees and 70 degrees.

Calculation of the snow load for a typical structure based on IBC and ASCE 7 provisions is illustrated in Example 2.6. This example also illustrates the effects of using a load on a horizontal plane in design calculations.

EXAMPLE 2.6 Snow Loads Using ASD

Assuming that the basic design snow load is not specified by the local building official, determine the total design dead load plus snow load for the rafters in the building illustrated in Figure 2.6A. The building is a standard residential occupancy located near Houghton, Michigan in surface roughness C terrain, with trees providing shelter on all sides of the structure. The building is heated, the rafters are sloped at 6 in./ft, and the roof covering consists of cement asbestos shingles. Determine the shear and moment for the rafters under dead plus snow loads if they are spaced 4 ft-0 in. o.c. Roof dead load D has been estimated as 14 psf along the roof surface.

Snow Load

Ground snow load:	$p_g = 80$ psf	from ASCE 7 Figure 7.2-1
Importance factor:	$I_s = 1.0$	from ASCE 7 Table 1.5-1 Risk Category II
Snow exposure factor:	$C_e = 1.1$	from ASCE 7 Table 7.3-1 for "sheltered" roof

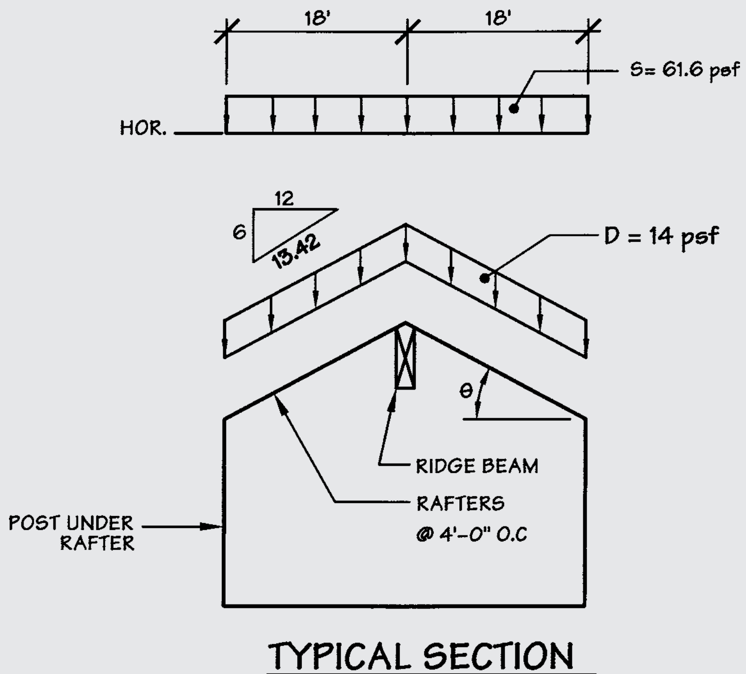

FIGURE 2.6A Example building section for roof loads.

Thermal factor:	$C_t = 1.0$	from ASCE 7 Table 7.3-2
Roof slope factor:	$C_s = 1.0$	from ASCE 7 Figure 7.4-1A at 6:12 roof slope
Design snow load:		

$$
\begin{aligned}
S &= p_s = C_s\,p_f \\
&= C_s\,0.7C_e\,C_t\,I_s\,p_g \\
&= (1.0)(0.7)(1.1)(1.0)(1.0)(80) = 61.6 \text{ psf}
\end{aligned}
$$

Combining Loads

In computing the combined load to the rafters in the roof, the different lengths of the dead and snow loads must be taken into account. In addition, the shear and moment in the rafters may be combined using the sloping beam method or the horizontal plane method. In the *sloping beam method*, the gravity load is resolved into components that are parallel and perpendicular to the member. The values of shear and moment are based on the normal (perpendicular) component of load and a span length equal to the full length of the rafter. In the *horizontal plane method*, the gravity load is applied to a beam with a span that is taken as the horizontal projection of the rafter. Both methods are illustrated, and the maximum values of shear and moment are compared (see Figure 2.6B).

$$
\begin{aligned}
\text{TL} &= \text{D} + \text{S} \\
&= 14 + 61.6\left(\frac{18}{20.12}\right) \\
&= 69 \text{ psf} \\
w &= 69 \text{ psf} \times 4 \text{ ft} \\
&= 276 \text{ lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
\text{TL} &= \text{D} + \text{S} \\
&= 14\left(\frac{20.12}{18}\right) + 61.6 \\
&= 77.2 \text{ psf} \\
w &= 77.2 \text{ psf} \times 4 \text{ ft} \\
&= 309 \text{ lb/ft}
\end{aligned}
$$

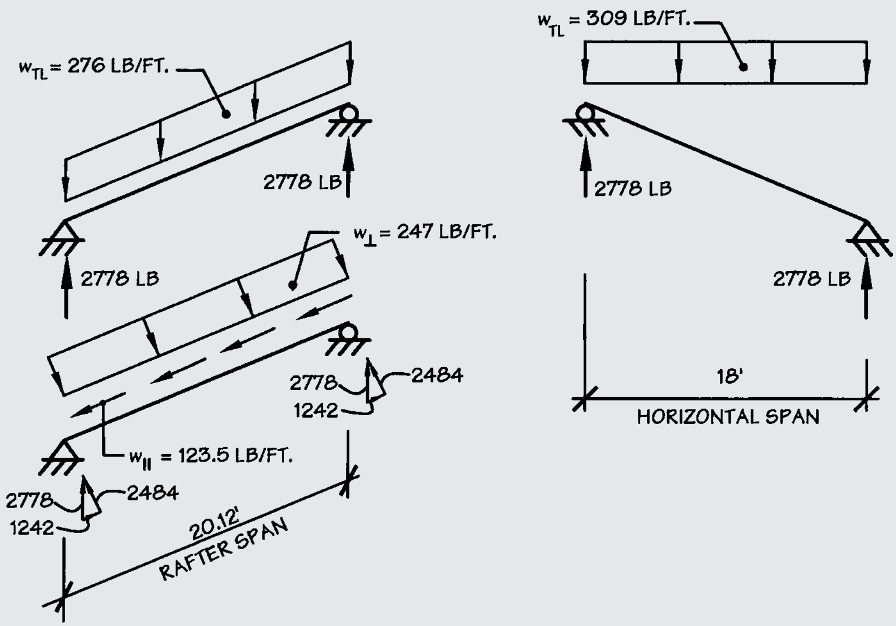

Figure 2.6b Comparison of *sloping beam method* (left) and *horizontal plane method* (right) for determining shears and moments in an inclined beam.

Use load normal to roof and rafter span parallel to roof.

$$V = \frac{wL}{2} = \frac{0.247(20.12)}{2}$$
$$= 2.48 \text{ k}$$
$$M = \frac{wL^2}{8} = \frac{0.247(20.12)^2}{8}$$
$$= 12.5 \text{ ft-k}$$

Use total vertical load and projected horizontal span.

$$V = \frac{wL}{2} = \frac{0.309(18)}{2}$$
$$= 2.48 \text{ k} \ \text{(conservative)}$$
$$M = \frac{wL^2}{8} = \frac{0.309(18)^2}{8}$$
$$= 12.5 \text{ ft-k} \ \text{(same)}$$

NOTE: The horizontal plane method is commonly used in practice to calculate design values for inclined beams such as rafters. This approach is convenient and gives equivalent design moments and conservative values for shear compared with the sloping beam analysis. (By definition *shear* is an internal force *perpendicular* to the longitudinal axis of a beam. Therefore, the calculation of shear using the sloping beam method in this example is theoretically correct.)

In addition to these basic guidelines for snow loads on flat or sloped roofs, ASCE 7 provides more comprehensive procedures for evaluating snow loads under special conditions. For example, ASCE 7 provisions include consideration of drifting snow and unbalanced snow loads, sliding snow from higher roof surfaces, rain-on-snow surcharge loads for flat roofs, and minimum design snow loads for low-slope roofs (slope ≤ 5 degrees).

2.5 Soil Loads and Hydrostatic Pressure

The notation H is used for lateral soil loads, loads due to hydrostatic pressure, and the pressure of bulk materials. Soil lateral loads and hydrostatic pressure are introduced in ASCE 7 Sec. 3.2 and IBC Sec. 1610.

Soil lateral loading most commonly occurs at retaining walls. It is relatively unusual for wood to be directly loaded by retained soils. One notable exception to this is permanent wood foundations, used in some regions of the United States. While soil lateral loading will most often come from a geotechnical investigation report, ASCE 7 Table 3.2-1 provides design lateral soil pressures (in psf, per foot of soil depth) for a range of soil classifications. Where retaining walls are provided, it is possible to develop hydrostatic pressure in addition. Hydrostatic pressure is most often avoided by providing drains behind retaining walls. In cases where it is not possible to provide drains, design for combined soil and hydrostatic lateral pressures is required.

In conditions where hydrostatic lateral pressures can develop, it is possible to also have upward hydrostatic pressures on adjacent floor slabs. These upward pressures would also use the notation H in load combinations.

The notation H is also defined in ASCE 7 and the IBC to include pressure of bulk materials. Although no discussion of this use is provided, it is thought to include pressure due to storage of grain, aggregates, or other bulk solids that can exert lateral pressures.

2.6 Loads Due to Fluids

The notation F is used for loads due to fluids with well-defined pressures and maximum heights. There is no specific discussion of this use in either ASCE 7 or IBC. It is clearly not intended to address flood loads or hydrostatic pressure, as these are covered in other load types. This leaves other fluids that might occur in building structures.

Where fluids are contained in nonbuilding structures or nonstructural components, the reader is cautioned that a wide range of other standards might be applicable. ASCE 7 Chaps. 13 and 15 may provide some guidance for these types of structures.

2.7 Rain Loads

The notation R is used for rain loads. Rain loads are discussed in ASCE 7 Chap. 8 and IBC Sec. 1611. The concept of rain load is primarily applicable to low slope roofs. Where this is the case, internal drainage systems are generally provided to collect rainwater falling on the roof. It is also generally required that this type of roof have secondary drains or scuppers to provide drainage in case the primary drainage system is not operable. The rain load R is calculated as

$$R = 5.2\,(d_s + d_h)$$

where R = rain load in psf
5.2 = weight of water per inch
d_s = the depth of water at the inlet to the secondary drainage system, based on an undeflected roof
d_h = the additional depth of hydraulic head that develops at the inlet at its design flow

Where roof deflection might result in additional water weight due to ponding, this must also be considered. New in the 2016 edition of ASCE 7 is the discussion of susceptible bays. This is added to draw attention to potential ponding instability. This occurs when deflection under rain loads allows the rain loads to further increase. In very flexible roof systems the increasing rain load can lead to overloading of the roof framing and possible member failure.

While the possible weight of water on the roof may seem small, every year roof collapses occur during heavy rain storms, attesting to the importance of design for rain water. This load type need not be considered for sloped roofs that cannot develop water buildup.

2.8 Flood Loads

The notation F_a is used for flood loads. Flood loads are addressed in ASCE 7 Chap. 5 and IBC Sec. 1612. The provisions in IBC Sec. 1612 introduce terminology used in design for flood loads, provide direction to the jurisdiction regarding establishment of flood hazard areas, identify needed flood hazard documentation, and refer the user to ASCE 24 (Ref. 2.6) for design and construction requirements.

Because explanation of design for flood loads requires some amount of detail, and because wood structures themselves will generally be elevated above design flood elevations rather than being designed to resist flood loading, this book will not describe details of flood loads. The reader is referred to ASCE 7 Chap. 5 and ASCE 24 for information.

2.9 Tsunami Loads

New in the 2016 Edition of ASCE 7 and the 2018 IBC are provisions for Tsunami Loads. Recent tsunami events, including the 2004 Indian Ocean and 2011 Tohoku Earthquakes and Tsunamis, have drawn attention to the need to consider this hazard. This new chapter is the result of significant efforts to develop mitigation measures that can reasonably be incorporated into building design. Key elements of the chapter are mapping of tsunami hazard areas, methods to identify the horizontal inundation limit and inundation depth, and measures to determine tsunami design forces from this information. The extreme forces that occur with tsunami water and debris impact loading are beyond what most ordinary structures can withstand. For this reason, focus is primarily on design of structures assigned to Tsunami Risk Categories III and IV. Design for tsunami loading is an advanced topic that is beyond the scope of this text. The reader is referred to ASCE 7 and commentary for further information.

2.10 Self-Straining Loads

The notation T is used for self-straining loads. Discussion of self-straining loads is found in the lengthy description of the notation in ASCE 7 Chapter 2. It is: "Self-straining force arising from contraction or expansion resulting from environmental or operational temperature change, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof." ASCE 7 Sec. 2.4.4 discusses combining T with other loads.

Self-straining loads primarily occur due to dimensional change of the structural element itself, or movement of the support for the member. Where members are free to move with dimensional or support change, forces do not develop. Where elements are restrained against movement, dimensional or support change will induce internal stresses. One example of this behavior is the shrinkage and shortening of post-tensioned concrete slab and beam systems; where the structural system permits unrestrained shrinkage, minimal force will result. Where the structural system restrains free shrinkage, significant forces can result, which if not accounted for can damage structures. Another example is the temperature-driven expansion and contraction of aluminum mullions for curtain wall systems; where not properly allowed for, mullion connections to the structure have been failed by self-straining forces T. It is unusual for self-straining forces to need to be considered in design of wood structures.

2.11 Wind Loads—Introduction

The notation W is used for wind loads. Wind loads are addressed in ASCE 7 Chaps. 26 to 31 and IBC Sec. 1609. The IBC refers the user to ASCE 7 provisions for determination of wind loads on buildings; however, a number of wind load provisions are presented in IBC Sec. 1609. These include identification of prescriptive design provisions that are permitted to be used in lieu of ASCE 7 wind provisions, requirements for opening protection in hurricane prone areas vulnerable to windborne debris, definitions, information drawn from ASCE 7 to permit identification of basic wind speed and exposure category, and requirements for wind design of roofing systems. The requirements for windborne debris should be considered by the designer early in the design process for buildings in hurricane prone locations, because doors and windows not properly protected can be breached in high wind events, allowing wind to enter the building and creating higher wind loading on the structure. The discussion in this section will be focused on ASCE 7 provisions.

ASCE 7 provisions are based on the results of extensive research regarding wind loads on structures and components of various sizes and configurations in a wide variety of simulated exposure conditions. ASCE 7 presents a number of different ways to determine wind loads on structures. First, the designer can select between determining the loads based on a calculation method or using wind tunnel modeling based procedures. The wind tunnel procedure is used for complex buildings that might be anticipated to have unusual dynamic behavior; use is limited to a small group of buildings for which the time and expense of a detailed study can be justified. Wind tunnel procedures are addressed in Chap. 31 of ASCE 7.

For the majority of buildings, wind loads will be determined using a calculated method. Within the method, two broad categories of loading are considered: loading to the main wind force-resisting system (MWFRS) (most often floor and roof diaphragms and shear walls) and loading to components and cladding (e.g., wall studs and roof rafters). These categories will be explained in more detail in the following sections. For the MWFRS loading, ASCE 7 addresses the Directional Procedure in Chap. 27 and further breaks this procedure into Part 1, addressing all building heights, and Part 2 for buildings 160 feet in height or less. Similarly ASCE 7 addresses the Envelope Procedure in Chap. 28, and further breaks this procedure into Part 1 that addresses all enclosed and partially enclosed low-rise buildings, and Part 2 that is limited to simple diaphragm buildings.

ASCE 7 Chap. 29 addresses wind loads on the MWFRS of other structure types and building appurtenances. Common uses of this chapter include wind design of signs and fences. Finally, Chap. 30 addressed wind loads for design of components and cladding. Four parts to Chap. 30 address closed, partially enclosed, or open buildings using general and simplified methods.

Two very significant changes occurred in the wind design provisions of ASCE 7-10. The first was the reorganization of the wind design provisions. Previously contained in Chap. 6 of ASCE 7, the wind provisions are now in Chaps. 26 to 31, as just discussed.

The second major change was in the wind velocity used for design, which has been changed from a velocity representing ASD level design loads to a velocity representing strength (LRFD) level design loads. This change will be discussed further in later sections, as will additional changes in ASCE 7-16.

This book will illustrate the use of Chap. 28, Part 2 for the MWFRS, the method for enclosed, simple-diaphragm low-rise buildings. This book will also demonstrate the use of Chap. 30, Part 2 for components and cladding, the simplified method for low-rise buildings. After following the discussion of these methods in this book, the reader is encouraged to look at the use of the other ASCE 7 methods, particularly Chap. 27, Part 2. An extensive commentary to the ASCE 7 wind design provision is offered in the ASCE 7 commentary. The reader is encouraged to use this resource.

ASCE 7 places a series of limitations on the use of Chap. 28, Part 2, which can be found in Sec. 28.5.2. These limitations can be better understood once a few terms are introduced. The MWFRS is a system providing wind resistance for the overall structure. In wood-frame buildings, the MWFRS most commonly consists of shearwalls (sheathed walls that resist in-plane loads) and roof and floor diaphragms (sheathed floor and roof assemblies that transmit in-plane loads to the shearwalls). This book uses the term MWFRS when discussing wind loads only, and the term *lateral-force-resisting system* (LFRS) when discussing resistance to both wind and seismic forces. We will see later in this chapter that the MWFRS is generally also the seismic-force resisting system. *Components and cladding* are the members making up the exterior envelope of the building, including wall and roof framing, sheathing, and finish materials. ASCE 7 addresses limitations for the MWFRS and components and cladding separately.

In order for the MWFRS to be designed using Chap. 28, Part 2, the following conditions from Sec. 28.5.2 must be met:

1. It is a simple diaphragm building—a building in which both windward and leeward loads are transmitted through the floor and roof diaphragms to the same MWFRS (Sec. 26.2).
2. The building is low rise—it has a mean roof height less than 60 ft, and has a least horizontal dimension not less than the mean roof height (Sec. 26.2).
3. The building is enclosed—and meets requirements for wind-borne debris protection, if applicable (Sec. 26.12.3).
4. The building is regular—has no unusual geometrical irregularity in spatial form (Sec. 26.2).
5. The building is not classified as a flexible building—has a fundamental frequency greater than 1 hertz (fundamental period less than 1 second) (Sec. 26.2).
6. The building does not have response characteristics that create unusual loading (such as galloping or vortex shedding) and is not sited in a location where unusual wind load effects might occur.
7. The building has an approximately symmetrical cross section in each direction, and has a flat roof or a gable or hip roof with slope less than or equal to 45 degrees (12 in 12 pitch).
8. The building is exempted from the torsional load cases of ASCE 7 Figure 28.4-1 (per footnote 5, one- and two-story wood-frame construction is exempted), or the torsional load cases do not control design.

While this is a rather daunting list of limitations, Chap. 28, Part 2 can still be applied to most one and two-story wood-frame buildings, in most locations. Buildings with large variations in the amount of opening in different exterior walls should be carefully evaluated to see if they qualify as enclosed buildings per the ASCE definitions. Also, checking of torsional load cases will be necessary for buildings with three or more stories.

Before proceeding to discuss the wind pressure equations, two parameters affecting wind pressures should be discussed. These are the building enclosure classification and the exposure category.

For purposes of wind design, structures are designated as open, partially enclosed, partially open, and enclosed. These categories are described in ASCE 7 Sec. 26.2 definitions, and can notably affect the wind loads that can develop on a structure. The most critical wind condition comes with a partially enclosed categorization, in which one side of the structure has significant openings through which wind can enter, and the other side is substantially closed. This causes the wind to stagnate against the far wall and causes high internal wind pressures that act outward on all walls and upward on the roof. The most common designation for occupied buildings will be enclosed, because windows and doors are not required to be considered openings provided they are designed for wind load and, in windborne debris areas, have windborne debris protection. Note that provisions in ASCE 7 that are applicable to enclosed buildings are also applicable to partially open buildings. This clarification was not included in the first printing of ASCE 7-16.