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U.S. Department of Housing and Urban Development
Office of Policy Development and Research

Residential Structural Design Guide: 2000 Edition

A State-of-the-Art Review and Application of Engineering Information for Light-Frame Homes, Apartments, and Townhouses

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Residential Structural Design Guide: 2000 Edition

A State-of-the-Art Review and Application of Engineering Information for Light-Frame Homes, Apartments, and Townhouses

Prepared for

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Foreword

The increasing complexity of homes, the use of innovative materials and technologies, and the increased population in high-hazard areas of the United States have introduced many challenges to the building industry and design profession as a whole. These challenges call for the development and continual improvement of efficient engineering methods for housing applications as well as for the education of designers in the uniqueness of housing as a structural design problem.

This text is an initial effort to document and improve the unique structural engineering knowledge related to housing design and performance. It compliments current design practices and building code requirements with value-added technical information and guidance. In doing so, it supplements fundamental engineering principles with various technical resources and insights that focus on improving the understanding of conventional and engineered housing construction. Thus, it attempts to address deficiencies and inefficiencies in past housing construction practices and structural engineering concepts through a comprehensive design approach that draws on existing and innovative engineering technologies in a practical manner. The guide may be viewed as a "living document" subject to further improvement as the art and science of housing design evolves.

We hope that this guide will facilitate and advance efficient design of future housing whether built in conformance with prescriptive (i.e., "conventional") practices or specially engineered in part or whole. The desired effect is to continue to improve the value of American housing in terms of economy and structural performance.

Susan M. Wachter
Assistant Secretary for Policy
Development and Research

Preface

This document is a unique and comprehensive tool for design professionals, particularly structural engineers, seeking to provide value-added services to the producers and consumers of American housing. As such, the guide is organized around the following major objectives:

- to present a sound perspective on American housing relative to its history, construction characteristics, regulation, and performance experience;
- to provide the latest technical knowledge and engineering approaches for the design of homes to complement current code-prescribed design methods;
- to assemble relevant design data and methods in a single, comprehensive format that is instructional and simple to apply for the complete design of a home; and
- to reveal areas where gaps in existing research, design specifications, and analytic tools necessitate alternative methods of design and sound engineering judgment to produce efficient designs.

This guide consists of seven chapters. The layout and application of the various chapters are illustrated in the figure on page vii. Chapter 1 describes the basic substance of American housing, including conventional construction practices, alternative materials, building codes and standards, the role of design professionals, and actual experience with respect to performance problems and successes, particularly as related to natural hazards such as hurricanes and earthquakes. Chapter 2 introduces basic engineering concepts regarding safety, load path, and the structural system response of residential buildings, subassemblies, and components to various types of loads. Chapter 3 addresses design loads applicable to residential construction. Chapters 4 and 5 provide step-by-step design procedures for the various components and assemblies comprising the structure of a home—from the foundation to the roof. Chapter 6 is devoted to the design of light-frame homes to resist lateral loads from wind and earthquakes. Chapter 7 addresses the design of various types of connections in a wood-framed home that are important to the overall function of the numerous component parts. As appropriate, the guide offers additional resources and references on the topics addressed.

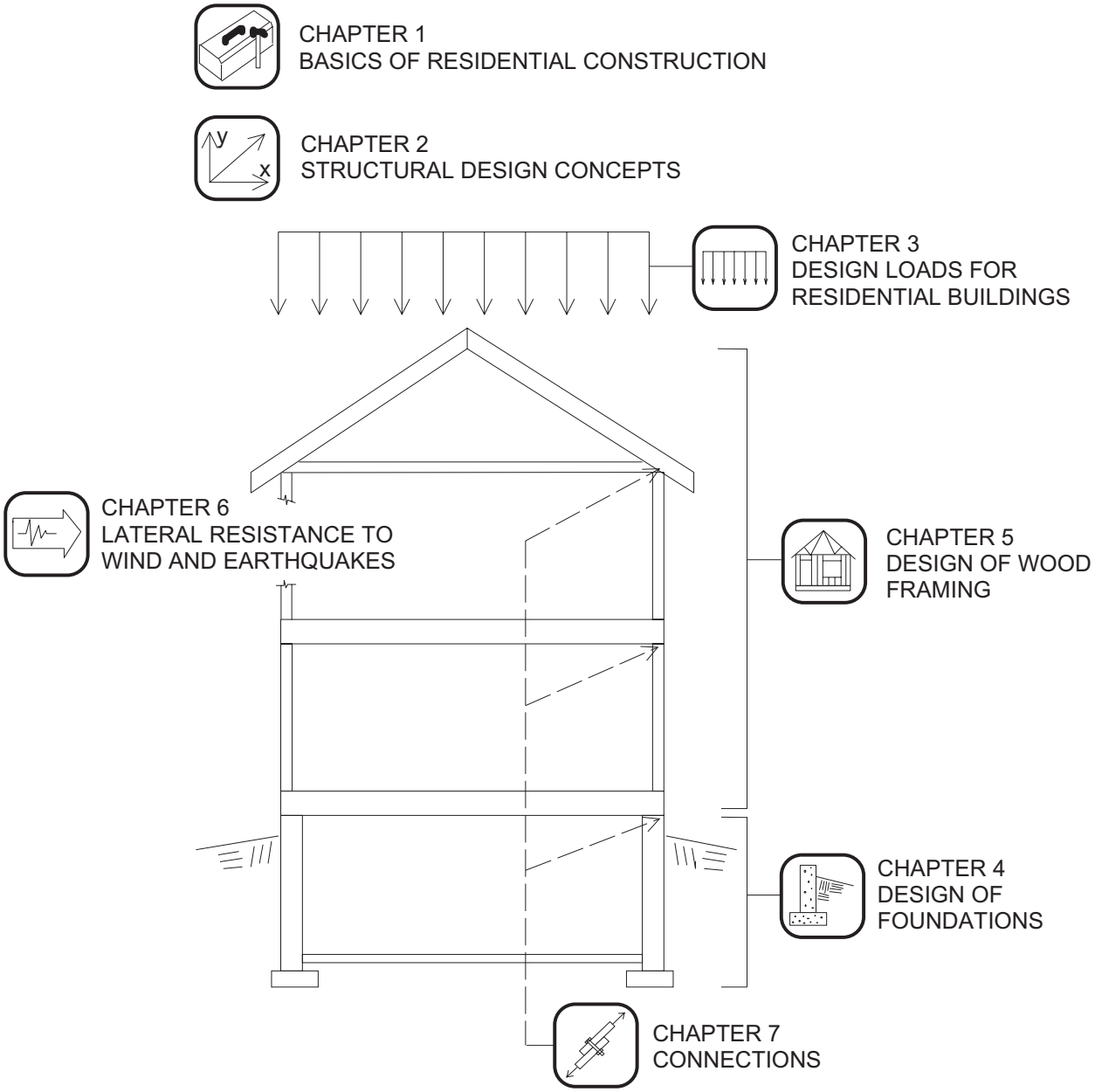
Given that most homes in the United States are built with wood structural materials, the guide focuses on appropriate methods of design associated with wood for the above-grade portion of the structure. Concrete or masonry are generally assumed to be used for the below-grade portion of the structure, although preservative-treated wood may also be used. Other materials and systems using various innovative approaches are considered in abbreviated form as appropriate. In some cases, innovative materials or systems can be used to address specific issues in the design and performance of homes. For example, steel framing is popular in Hawaii partly because of wood's special

problems with decay and termite damage. Likewise, partially reinforced masonry construction is used extensively in Florida because of its demonstrated ability to perform in high winds.

For typical wood-framed homes, the primary markets for engineering services lie in special load conditions, such as girder design for a custom house; corrective measures, such as repair of a damaged roof truss or floor joist; and high-hazard conditions such as on the West Coast (earthquakes) and the Gulf and Atlantic coasts (hurricanes). The design recommendations in the guide are based on the best information available to the authors for the safe and efficient design of homes. Much of the technical information and guidance is supplemental to building codes, standards, and design specifications that define current engineering practice. In fact, current building codes may not explicitly recognize some of the technical information or design methods described or recommended in the guide. Therefore, a competent professional designer should first compare and understand any differences between the content of this guide and local building code requirements. Any actual use of this guide by a competent professional may require appropriate substantiation as an "alternative method of analysis." The guide and references provided herein should help furnish the necessary documentation.

The use of alternative means and methods of design should not be taken lightly or without first carefully considering the wide range of implications related to the applicable building code’s minimum requirements for structural design, the local process of accepting alternative designs, the acceptability of the proposed alternative design method or data, and exposure to liability when attempting something new or innovative, even when carried out correctly. It is not the intent of this guide to steer a designer unwittingly into non-compliance with current regulatory requirements for the practice of design as governed by local building codes. Instead, the intent is to provide technical insights into and approaches to home design that have not been compiled elsewhere but deserve recognition and consideration. The guide is also intended to be instructional in a manner relevant to the current state of the art of home design.

Finally, it is hoped that this guide will foster a better understanding among engineers, architects, building code officials, and home builders by clarifying the perception of homes as structural systems. As such, the guide should help structural designers perform their services more effectively and assist in integrating their skills with others who contribute to the production of safe and affordable homes in the United States.



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CHAPTER 1

Basics of Residential Construction

1.1 Conventional Residential Construction

The conventional American house has been shaped over time by a variety of factors. Foremost, the abundance of wood as a readily available resource has dictated traditional American housing construction, first as log cabins, then as post-and-beam structures, and finally as light-frame buildings. The basic residential construction technique has remained much the same since the introduction of light wood-framed construction in the mid-1800s and is generally referred to as conventional construction. See Figures 1.1a through 1.1c for illustrations of various historical and modern construction methods using wood members.

In *post-and-beam framing*, structural columns support horizontal members. Post-and-beam framing is typified by the use of large timber members. Traditional *balloon framing* consists of closely spaced light vertical structural members that extend from the foundation sill to the roof plates. *Platform framing* is the modern adaptation of balloon framing whereby vertical members extend from the floor to the ceiling of each story. Balloon and platform framings are not simple adaptations of post-and-beam framing but are actually unique forms of wood construction. Platform framing is used today in most wood-framed buildings; however, variations of balloon framing may be used in certain parts of otherwise platform-framed buildings, such as great rooms, stairwells, and gable-end walls where continuous wall framing provides greater structural integrity. Figure 1.2 depicts a modern home under construction.

FIGURE 1.1a Post-and-Beam Construction (Historical)

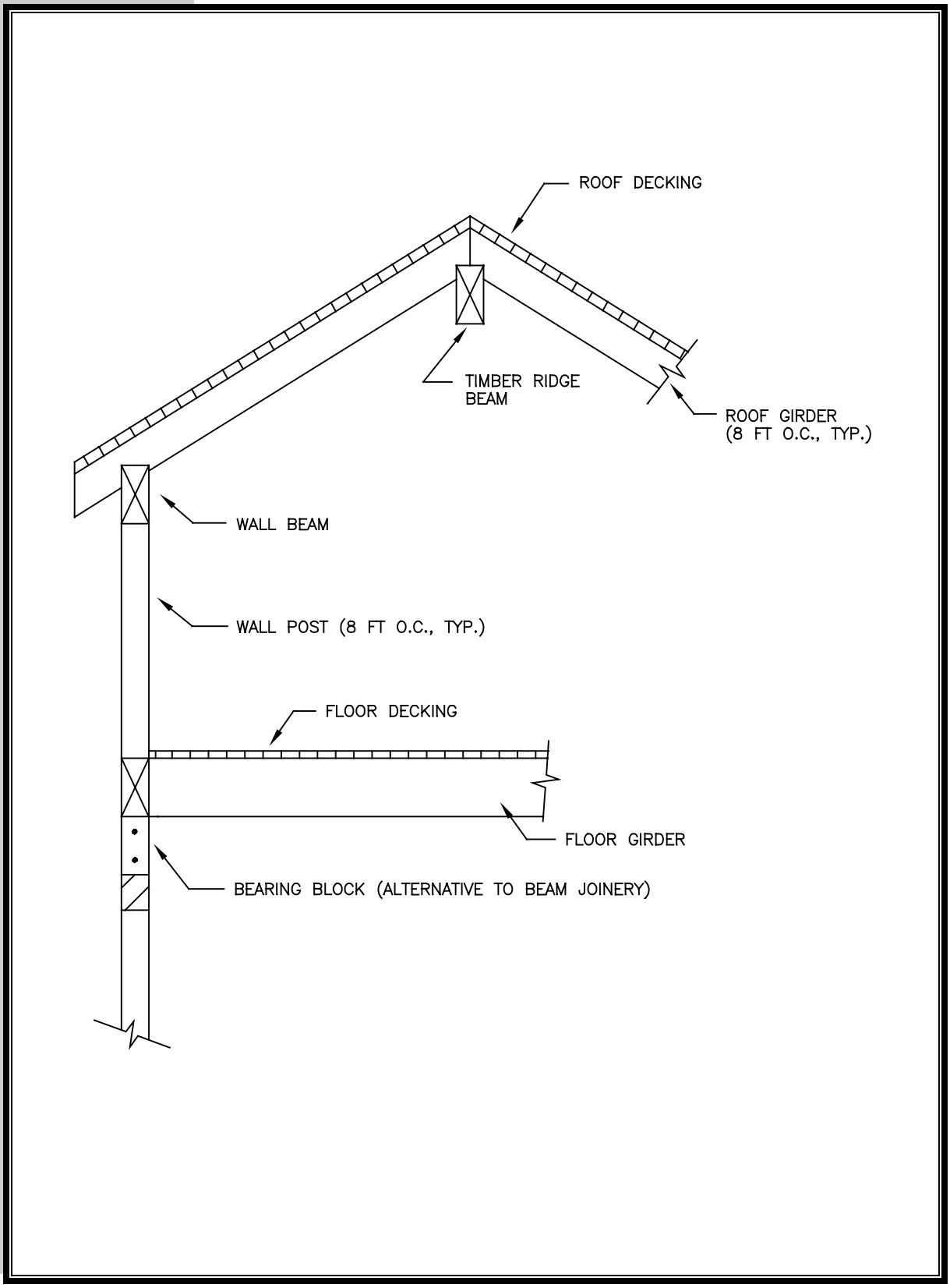
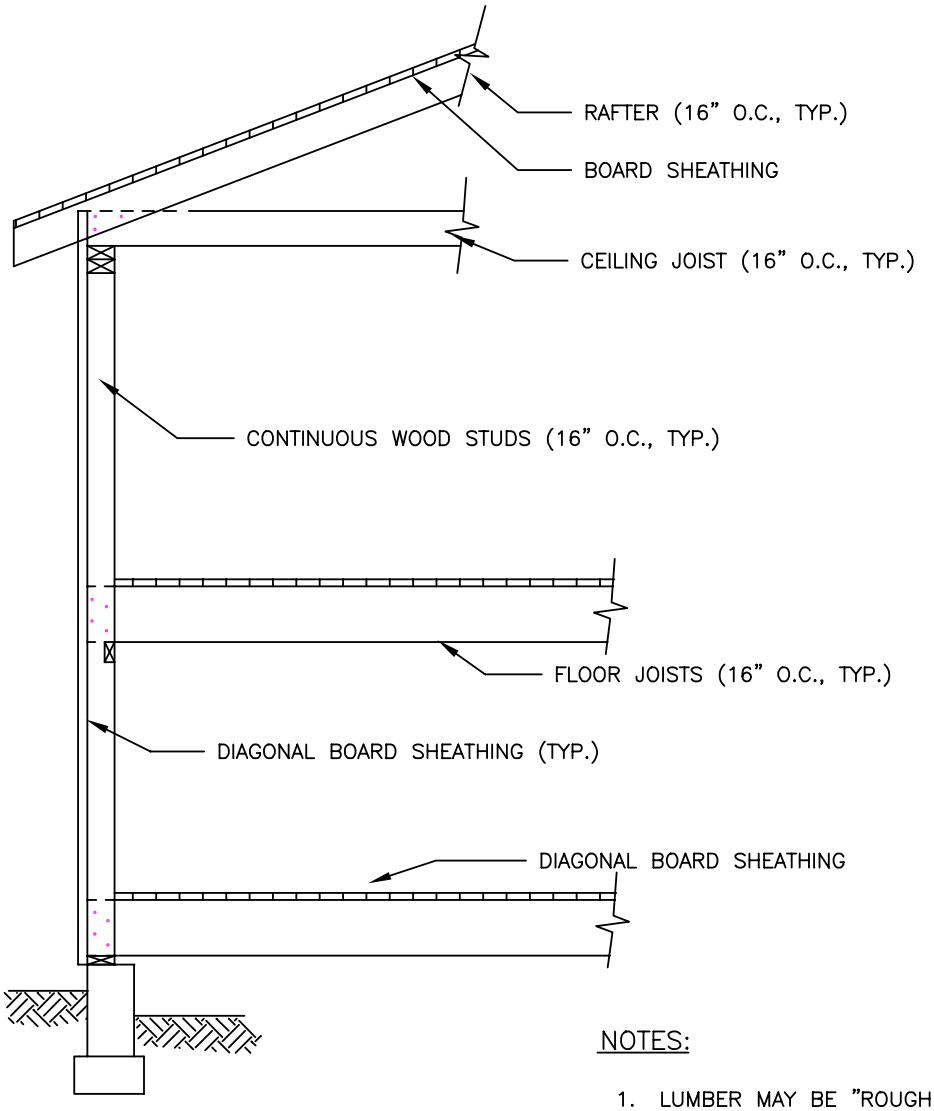


FIGURE 1.1b Balloon-Frame Construction (Historical)



NOTES:

1. LUMBER MAY BE "ROUGH SAWN" 2 TO 3 INCHES THICK IN OLDER BUILDINGS.
2. RAFTERS AND JOISTS MAY BE SMALL DIAMETER WOOD POLES INSTEAD OF SAWN LUMBER.

FIGURE 1.1c Modern Platform-Frame Construction

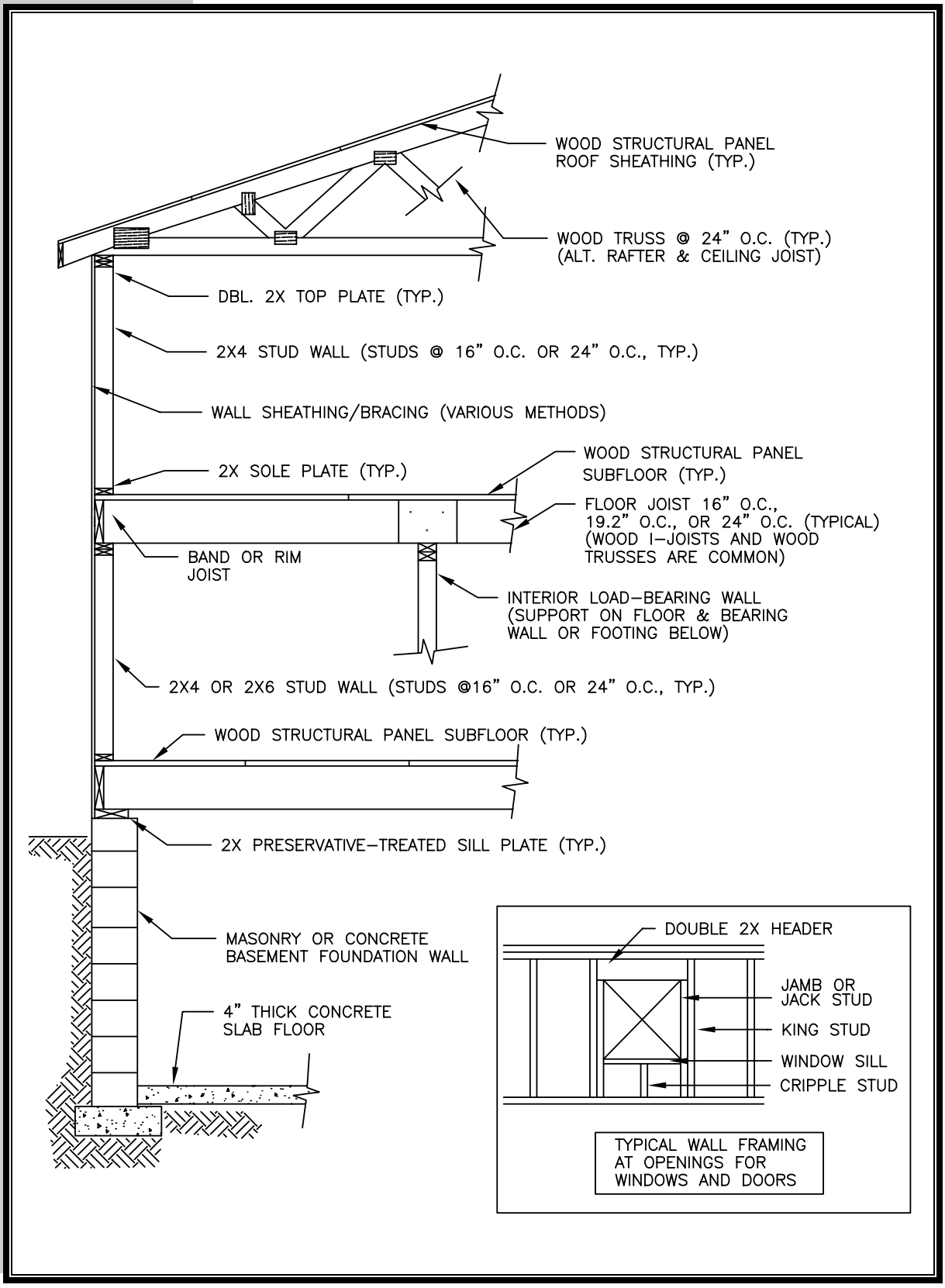


FIGURE 1.2 Modern Platform-Framed House under Construction

Conventional or prescriptive construction practices are based as much on experience as on technical analysis and theory (HEW, 1931). When incorporated into a building code, prescriptive (sometimes called "cook book") construction requirements can be easily followed by a builder and inspected by a code official without the services of a design professional. It is also common for design professionals, including architects and engineers, to apply conventional practice in typical design conditions but to undertake special design for certain parts of a home that are beyond the scope of a prescriptive residential building code. Over the years, the housing market has operated efficiently with minimal involvement of design professionals. Section 1.5 explores the current role of design professionals in residential construction as well as some more recent trends.

While dimensional lumber has remained the predominant material used in twentieth-century house construction, the size of the material has been reduced from the rough-sawn, 2-inch-thick members used at the turn of the century to today's nominal "dressed" sizes with actual thickness of 1.5 inches for standard framing lumber. The result has been significant improvement in economy and resource utilization, but not without significant structural trade-offs in the interest of optimization. The mid- to late 1900s have seen several significant innovations in wood-framed construction. One example is the development of the metal plate-connected wood truss in the 1950s. Wood truss roof framing is now used in most new homes because it is generally more efficient than older stick-framing methods. Another example is plywood structural sheathing panels that entered the market in the 1950s and quickly replaced board sheathing on walls, floors, and

roofs. Another engineered wood product known as oriented strand board (OSB) is now substantially replacing plywood.

In addition, it is important to recognize that while the above changes in materials and methods were occurring, significant changes in house design have continued to creep into the residential market in the way of larger homes with more complicated architectural features, long-span floors and roofs, large open interior spaces, and more amenities. Certainly, the collective effect of the above changes on the structural qualities of most homes is notable.

The references below are recommended for a more in-depth understanding of conventional housing design, detailing, and construction. Section 1.8–References–provides detailed citations.

- *Wood Frame House Construction*, Second Edition (NAHB, 1992)
- *Cost-Effective Home Building: A Design and Construction Handbook* (NAHB, 1994)
- *Modern Carpentry–Building Construction Details in Easy-to-Understand Form*, Seventh Edition (Wagner, 1992)
- *International One- and Two-Family Dwelling Code* (ICC, 1998)

The following structural design references are also recommended for use with Chapters 3 through 7 of this guide:

- NDS–*National Design Specification for Wood Construction and Supplement* (AF&PA, 1997);
- ACI-318–*Building Code Requirements for Structural Concrete* (ACI, 1999);
- ACI-530–*Building Code Requirements for Masonry Structures* (ACI, 1999);
- ASCE 7-98–*Minimum Design Loads for Buildings and Other Structures* (ASCE, 1999); and
- local building code.

1.2 Industrialized Housing

Most homes in the United States are still site-built; that is, they follow a “stick framing” approach. With this method, wood members are assembled on site in the order of construction from the foundation up. The primary advantage of on-site building is flexibility in meeting variations in housing styles, design details, and changes specified by the owner or builder. However, an increasing number of today’s site-built homes use components that are fabricated in an off-site plant. Prime examples include wall panels and metal plate-connected wood roof trusses. The blend of stick-framing and plant-built components is referred to as "component building."

A step beyond component building is modular housing. Modular housing is constructed in essentially the same manner as site-built housing except that houses are plant-built in finished modules (typically two or more modules) and shipped to the jobsite for placement on conventional foundations. Modular

housing is built to comply with the same building codes that govern site-built housing. Generally, modular housing accounts for less than 10 percent of total production of single-family housing units.

Manufactured housing (also called mobile homes) is also constructed by using wood-framed methods; however, the methods comply with federal preemptive standards specified in the Code of Federal Regulations (HUD Code). This popular form of industrialized housing is completely factory-assembled and then delivered to a site by using an integral chassis for road travel and foundation support. In recent years, factory-built housing has captured more than 20 percent of new housing starts in the United States.

1.3 Alternative Materials and Methods

More recently, several innovations in structural materials have been introduced to residential construction. In fact, alternatives to conventional wood-framed construction are gaining recognition in modern building codes. It is important for designers to become familiar with these alternatives since their effective integration into conventional home building may require the services of a design professional. In addition, a standard practice in one region of the country may be viewed as an alternative in another and provides opportunities for innovation across regional norms.

Many options in the realm of materials are already available. The following pages describe several significant examples. In addition, the following contacts are useful for obtaining design and construction information on the alternative materials and methods for house construction discussed next:

General Contacts
HUD User (800-245-2691, www.huduser.org)
ToolBase (800-898-2842, www.nahbrc.org)

Engineered Wood Products
American Wood Council (800-292-2372, www.awc.org)
APA–The Engineered Wood Association (206-565-6600, www.apawood.org)
Wood Truss Council of America (608-274-4849, www.woodtruss.com)
Wood I-Joist Manufacturer's Association (www.i-joist.com)

Cold-Formed Steel
North American Steel Framing Alliance (202-785-2022, www.steelframingalliance.com)
American Iron and Steel Institute (1-800-898-2842, www.steel.org)
Light-Gauge Steel Engineer's Association (615-386-7139, www.lgsea.com)
Steel Truss & Component Association (608-268-1031, www.steeltruss.org)

Insulating Concrete Forms
Portland Cement Association (847-966-6200, www.portcement.org)
Insulating Concrete Form Association (847-657-9730, www.forms.org)

Masonry
National Concrete Masonry Association (703-713-1900, www.ncma.org)

Engineered wood products and components (see Figure 1.3) have gained considerable popularity in recent years. Engineered wood products and components include wood-based materials and assemblies of wood products with structural properties similar to or better than the sum of their component parts. Examples include metal plate-connected wood trusses, wood I-joists, laminated veneer lumber, plywood, oriented strand board, glue-laminated lumber, and parallel strand lumber. Oriented strand board (OSB) structural panels are rapidly displacing plywood as a favored product for wall, floor, and roof sheathing. Wood I-joists and wood trusses are now used in 31.5 and 12.5 percent, respectively, of the total framed floor area in all new homes each year (NAHBRC, 1998). The increased use of engineered wood products is the result of many years of research and product development and, more important, reflects the economics of the building materials market. Engineered wood products generally offer improved dimensional stability, increased structural capability, ease of construction, and more efficient use of the nation's lumber resources. And they do not require a significant change in construction technique. The designer should, however, carefully consider the unique detailing and connection requirements associated with engineered wood products and ensure that the requirements are clearly understood in the design office and at the jobsite. Design guidance, such as span tables and construction details, is usually available from the manufacturers of these predominantly proprietary products.

FIGURE 1.3 House Construction Using Engineered Wood Components

Cold-formed steel framing (previously known as light-gauge steel framing) has been produced for many years by a fragmented industry with nonstandardized products serving primarily the commercial design and construction market. However, a recent cooperative effort between industry and the U.S. Department of Housing and Urban Development (HUD) has led to the development of standard minimum dimensions and structural properties for basic cold-formed steel framing materials. The express purpose of the venture was to create prescriptive construction requirements for the residential market. Cold-formed steel framing is currently used in exterior walls and interior walls in about 1 and 7.6 percent, respectively, of annual new housing starts (NAHB, 1998). The benefits of cold-formed steel include cost, durability, light weight, and strength (NAHBRC, 1994; HUD, 1994). Figure 1.4 illustrates the use of cold-formed steel framing in a home. The construction method is detailed in *Prescriptive Method for Residential Cold-Formed Steel Framing, Second Edition* and has been adopted by the *International One- and Two-Family Dwelling Code* (HUD, 1997; ICC, 1998). It is interesting to note that a similar effort for residential wood-framed construction took place about 70 years ago (HEW, 1931).

FIGURE 1.4 House Construction Using Cold-Formed Steel Framing

Insulating concrete form (ICF) construction, as illustrated in Figure 1.5, combines the forming and insulating functions of concrete construction in a single step. While the product class is relatively new in the United States, it appears to be gaining acceptance. In a cooperative effort between industry and HUD, the product class was recently included in building codes after the establishment of minimum dimensions and standards for ICF concrete construction. The benefits of ICF construction include durability, strength, noise control, and energy efficiency (HUD, 1998). The method is detailed in *Prescriptive Method for Insulating Concrete Forms in Residential Construction* and has been adopted by the *Standard Building Code* (HUD, 1998; SBCCI, 1999). Additional building code recognition is forthcoming.

FIGURE 1.5 House Construction Using Insulating Concrete Forms

Concrete masonry construction, illustrated in Figure 1.6, is essentially unchanged in basic construction method; however, recently introduced products offer innovations that provide structural as well as architectural benefits. Masonry construction is well recognized for its fire-safety qualities, durability, noise control, and strength. Like most alternatives to conventional wood-framed construction, installed cost may be a local issue that needs to be balanced against other factors. For example, in hurricane-prone areas such as Florida, standard concrete masonry construction dominates the market where its performance in major hurricanes has been favorable when nominally reinforced using conventional practice. Nonetheless, at the national level, masonry above-grade wall construction represents less than 10 percent of annual housing starts.

FIGURE 1.6 ***House Construction Using Concrete Masonry***

1.4 Building Codes and Standards

Virtually all regions of the United States are covered by a legally enforceable building code that governs the design and construction of buildings, including residential dwellings. Although building codes are legally a state police power, most states allow local political jurisdictions to adopt or modify building codes to suit their "special needs" or, in a few cases, to write their own code. Almost all jurisdictions adopt one of the major model codes by legislative action instead of attempting to write their own code.

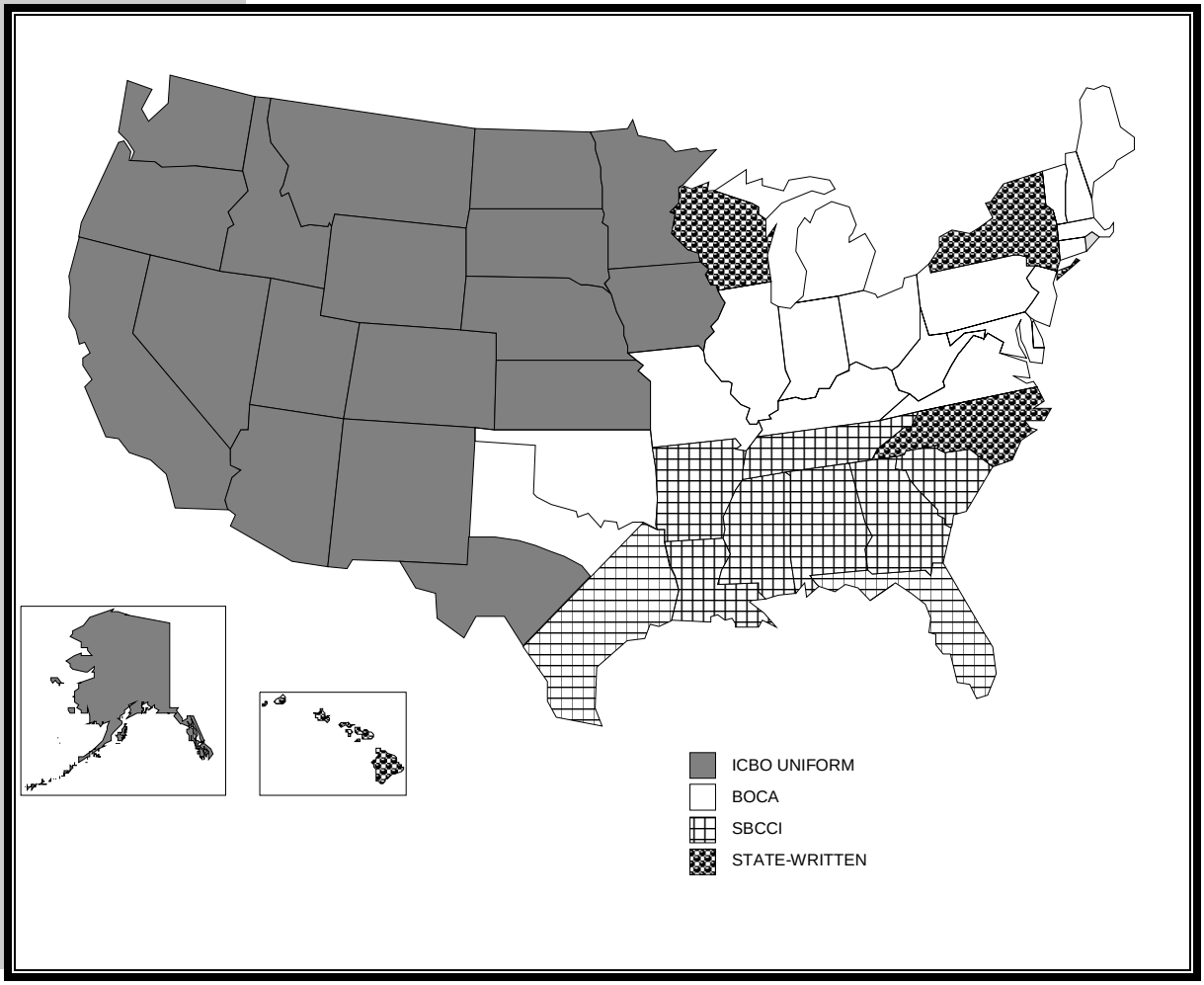
There are three major model building codes in the United States that are comprehensive; that is, they cover all types of buildings and occupancies–from a backyard storage shed to a high-rise office building or sports complex. The three major comprehensive building codes follow:

- National Building Code (NBC)
  Building Officials and Code Administrators International, Inc.
  4051 West Flossmoor Road
  Country Club Hills, IL 60478-5795
  708-799-2300
  www.bocai.org
- Standard Building Code (SBC)
  Southern Building Code Congress International, Inc.
  9800 Montclair Road
  Birmingham, AL 35213-1206
  205-591-1853
  www.sbcci.org

- Uniform Building Code (UBC)
  International Conference of Building Officials
  5360 Workman Mill Road
  Whittier, CA 90601-2298
  562-699-0541
  www.icbo.com

The three model codes are competitive in that they vie for adoption by state and local jurisdictions. In reality, however, the three codes are regional in nature, as indicated in Figure 1.7. Thus, the NBC tends to address conditions indigenous to the northeastern quarter of the United States (e.g., frost) while the SBC focuses on conditions in the southeastern quarter of the United States (e.g., hurricanes) and the UBC on conditions in the western half of the United States (e.g., earthquakes).

FIGURE 1.7 Use of Model Building Codes in the United States



To help resolve the problem of disunity among the three major building codes, the model building code organizations have recently entered into a joint effort (under the auspices of the International Code Council or ICC) to develop a single comprehensive building code called the International Building Code (IBC). The IBC is under development at the time of this writing. It draws heavily from the previous codes but adds new requirements for seismic design, wind design, stair geometry, energy conservation, and other vital subject areas. The new code is scheduled to be available in 2000, although several years may pass before change is realized on a national scale. In addition, another code-writing body, the National Fire Protection Association (NFPA), is developing a competitive model building code.

While the major model codes include some "deemed-to-comply" prescriptive requirements for conventional house construction, they focus primarily on performance (i.e., engineering) requirements for more complex buildings across the whole range of occupancy and construction types. To provide a comprehensive, easier-to-use code for residential construction, the three major code organizations participated in developing the *International One- and Two-Family Dwelling Code* (ICC, 1998), first published in 1971 as the *One- and Two-Family Dwelling Code* (OTFDC) by the Council of American Building Officials (CABO). Presented in logical construction sequence, the OTFDC is devoted entirely to simple prescriptive requirements for single-family detached and attached (townhouse) homes. Many state and local jurisdictions have adopted the OTFDC as an alternative to a major residential building code. Thus, designers and builders enjoy a choice as to which set of requirements best suits their purpose.

The major code organizations are also developing a replacement for the OTFDC in conjunction with the proposed IBC. Tentatively called the *International Residential Code for One- and Two-Family Dwellings* (IRC), it draws on earlier editions of the OTFDC and is slated for publication in 2000.

Model building codes do not provide detailed specifications for all building materials and products but rather refer to established industry standards, primarily those promulgated by the American Society for Testing and Materials (ASTM). Several ASTM standards are devoted to the measurement, classification, and grading of wood properties for structural applications as well as virtually all other building materials, including steel, concrete, and masonry. Design standards and guidelines for wood, steel, concrete materials, and other materials or applications are also maintained as reference standards in building codes. Currently, over 600 materials and testing standards are referenced in the building codes used in the United States.

For products and processes not explicitly recognized in the body of any of the model codes or standards, the model building code organizations provide a special code evaluation service with published reports. These evaluation reports are usually provided for a fee at the request of manufacturers. While the National Evaluation Service, Inc. (NES) provides a comprehensive evaluation relative to the three model codes mentioned above, each model code organization also performs evaluations independently for its specific code.

Seasoned designers spend countless hours in careful study and application of building codes and selected standards that relate to their area of practice. More important, these designers develop a sound understanding of the technical

rationale and intent behind various provisions in applicable building codes and design standards. This experience and knowledge, however, can become even more profitable when coupled with practical experiences from "the field." One of the most valuable sources of practical experience is the successes and failures of past designs and construction practices as presented in Section 1.6.

1.5 Role of the Design Professional

Since the primary user of this guide is assumed to be a design professional, it is important to understand the role that design professionals can play in the residential construction process, particularly with respect to recent trends. Design professionals offer a wide range of services to a builder or developer in the areas of land development, environmental impact assessments, geotechnical and foundation engineering, architectural design, structural engineering, and construction monitoring. This guide, however, focuses on two approaches to structural design as follows:

- Conventional design. Sometimes referred to as "nonengineered" construction, conventional design relies on standard practice as governed by prescriptive building code requirements for conventional residential buildings (see Section 1.4); some parts of the structure may be specially designed by an engineer or architect.
- Engineered design. Engineered design generally involves the application of conventions for engineering practice as represented in existing building codes and design standards.

Some of the conditions that typically cause concern in the planning and preconstruction phases of home building and thus sometimes create the need for professional design services are

- *structural configurations*, such as unusually long floor spans, unsupported wall heights, large openings, or long-span cathedral ceilings;
- *loading conditions*, such as high winds, high seismic risk, heavy snows, or abnormal equipment loads;
- *nonconventional building systems or materials*, such as composite materials, structural steel, or unusual connections and fasteners;
- *geotechnical or site conditions*, such as expansive soil, variable soil or rock foundation bearing, flood-prone areas, high water table, or steeply sloped sites; and
- *owner requirements*, such as special materials, appliance or fixture loads, atriums, and other special features.

The involvement of architects and structural engineers in the current residential market was recently studied. In a survey of 978 designers (594 architects and 384 structural engineers) in North America, at least 56 percent believed they were qualified to design buildings of four stories or less (Kozak and Cohen, 1999). Of this share, 80 percent noted that their workload was devoted to

buildings of four stories or less, with about 33 percent of that workload encompassing residential construction, including single-family dwellings, duplexes, multifamily units, and commercial/residential combinations.

While some larger production builders produce sufficient volume to justify an on-staff design professional, most builders use consultants on an as-needed basis. However, as more and more homes are built along the earthquake-prone West Coast and along the hurricane-prone Gulf and Atlantic seaboards, the involvement of structural design professionals seems to be increasing. Further, the added complexities of larger custom-built homes and special site conditions will spur demand for design specialists. Moreover, if nonconventional materials and methods of construction are to be used effectively, the services of a design professional are often required. In some instances, builders in high-hazard areas are using design professionals for on-site compliance inspections in addition to designing buildings.

The following organization may serve as a valuable on-demand resource for residential designers while creating better linkages with the residential building community and its needs:

REACH
Residential Engineer's and Architect's Council for Housing
NAHB Research Center, Inc.
800-898-2842
www.nahbrc.org

1.6 Housing Structural Performance

1.6.1 General

There are well over 100 million housing units in the United States, and approximately half are single-family dwellings. Each year, at least 1 million new single-family homes and townhomes are constructed, along with thousands of multifamily structures, most of which are low-rise apartments. Therefore, a small percent of all new residences may be expected to experience performance problems, most of which amount to minor defects that are easily detected and repaired. Other performance problems are unforeseen or undetected and may not be realized for several years, such as foundation problems related to subsurface soil conditions.

On a national scale, several homes are subjected to extreme climatic or geologic events in any given year. Some will be damaged due to a rare event that exceeds the performance expectations of the building code (i.e., a direct tornado strike or a large-magnitude hurricane, thunderstorm, or earthquake). Some problems may be associated with defective workmanship, premature product failure, design flaws, or durability problems (i.e., rot, termites, or corrosion). Often, it is a combination of factors that leads to the most dramatic forms of damage. Because the cause and effect of these problems do not usually fit simple generalizations, it is important to consider cause and effect objectively in terms of the overall housing inventory.

To limit the threat of life-threatening performance problems to reasonable levels, the role of building codes is to ensure that an acceptable level of safety is maintained over the life of a house. Since the public cannot benefit from an excessive degree of safety that it cannot afford, code requirements must also maintain a reasonable balance between affordability and safety. As implied by any rational interpretation of a building code or design objective, safety implies the existence of an acceptable level of risk. In this sense, economy or affordability may be broadly considered as a competing performance requirement. For a designer, the challenge is to consider optimum value and to use cost-effective design methods that result in acceptable performance in keeping with the intent or minimum requirements of the building code. In some cases, designers may be able to offer cost-effective options to builders and owners that improve performance well beyond the accepted norm.

1.6.2 Common Performance Issues

Objective information from a representative sample of the housing stock is not available to determine the magnitude and frequency of common performance problems. Instead, information must be gleaned and interpreted from indirect sources.

The following data are drawn from a published study of homeowner warranty insurance records in Canada (ONHWP/CMHC, 1994); similar studies are not easily found in the United States. The data do not represent the frequency of problems in the housing population at large but rather the frequency of various types of problems experienced by those homes that are the subject of an insurance claim. The data do, however, provide valuable insights into the performance problems of greatest concern–at least from the perspective of a homeowner warranty business.

Table 1.1 shows the top five performance problems typically found in Canadian warranty claims based on the frequency and cost of a claim. It may be presumed that claims would be similar in the United States since housing construction is similar, forgoing the difference that may be attributed to climate.

Considering the frequency of claim, the most common claim was for defects in drywall installation and finishing. The second most frequent claim was related to foundation walls; 90 percent of such claims were associated with cracks and water leakage. The other claims were primarily related to installation defects such as missing trim, poor finish, or sticking windows or doors.

In terms of cost to correct, foundation wall problems (usually associated with moisture intrusion) were by far the most costly. The second most costly defect involved the garage slab, which typically cracked in response to frost heaving or settlement. Ceramic floor tile claims (the third most costly claim) were generally associated with poor installation that resulted in uneven surfaces, inconsistent alignment, or cracking. Claims related to septic drain fields were associated with improper grading and undersized leaching fields. Though not shown in Table 1.1, problems in the above-grade structure (i.e., framing defects) resulted in about 6 percent of the total claims reported. While the frequency of structural related defects is comparatively small, the number is still significant in view of the total number of homes built each year. Even if many of the defects

may be considered nonconsequential in nature, others may not be and some may go undetected for the life of the structure. Ultimately, the significance of these types of defects must be viewed from the perspective of known consequences relative to housing performance and risk; refer to Sections 1.6.3 and 2.5.4.

TABLE 1.1 Top Five House Defects Based on Homeowner Warranty Claims

Based on Frequency of Claim	Based on Cost of Claim
1. Gypsum wall board finish	1. Foundation wall
2. Foundation wall	2. Garage slab
3. Window/door/skylight	3. Ceramic tiles
4. Trim and moldings	4. Septic drain field
5. Window/door/skylight frames	5. Other window/door/skylight

Source: Defect Prevention Research Project for Part 9 Houses (ONHWP/CMHC, 1994).

While the defects reported above are not necessarily related to building products, builders are generally averse to products that are "too new." Examples of recent class-action lawsuits in the United States give builders some reason to think twice about specifying new products such as

- Exterior Insulated Finish Systems (EIFS);
- fire-retardant treated plywood roof sheathing;
- certain composite sidings and exterior finishes; and
- polybutylene water piping.

It should be noted that many of these problems have been resolved by subsequent product improvements. Unfortunately, it is beyond the scope of this guide to give a complete account of the full range of problems experienced in housing construction.

1.6.3 Housing Performance in Hurricanes and Earthquakes

In recent years, scientifically designed studies of housing performance in natural disasters have permitted objective assessments of actual performance relative to that intended by building codes (HUD, 1993; HUD, 1994; HUD, 1998; HUD, 1999; NAHBRC, 1996). Conversely, anecdotal damage studies are often subject to notable bias. Nonetheless, both objective and subjective damage studies provide useful feedback to builders, designers, code officials, and others with an interest in housing performance. This section summarizes the findings from recent scientific studies of housing performance in hurricanes and earthquakes.

It is likely that the issue of housing performance in high-hazard areas will continue to increase in importance as the disproportionate concentration of development along the U.S. coastlines raises concerns about housing safety, affordability, and durability. Therefore, it is essential that housing performance is understood objectively as a prerequisite to guiding rational design and

construction decisions. Proper design that takes into account the wind and earthquake loads in Chapter 3 and the structural analysis procedures in Chapters 4, 5, 6, and 7 should result in efficient designs that address the performance issues discussed below. Regardless of the efforts made in design, however, the intended performance can be realized only with an adequate emphasis on installed quality. For this reason, some builders in high-hazard areas have retained the services of a design professional for on-site compliance inspections as well as for their design services. This practice offers additional quality assurance to the builder, designer, and owner in high-hazard areas of the country.

Hurricane Andrew

Without doubt, housing performance in major hurricanes provides ample evidence of problems that may be resolved through better design and construction practices. At the same time, misinformation and reaction following major hurricanes often produce a distorted picture of the extent, cause, and meaning of the damage relative to the population of affected structures. This section discusses the actual performance of the housing stock based on a damage survey and engineering analysis of a representative sample of homes subjected to the most extreme winds of Hurricane Andrew (HUD, 1998; HUD, 1993).

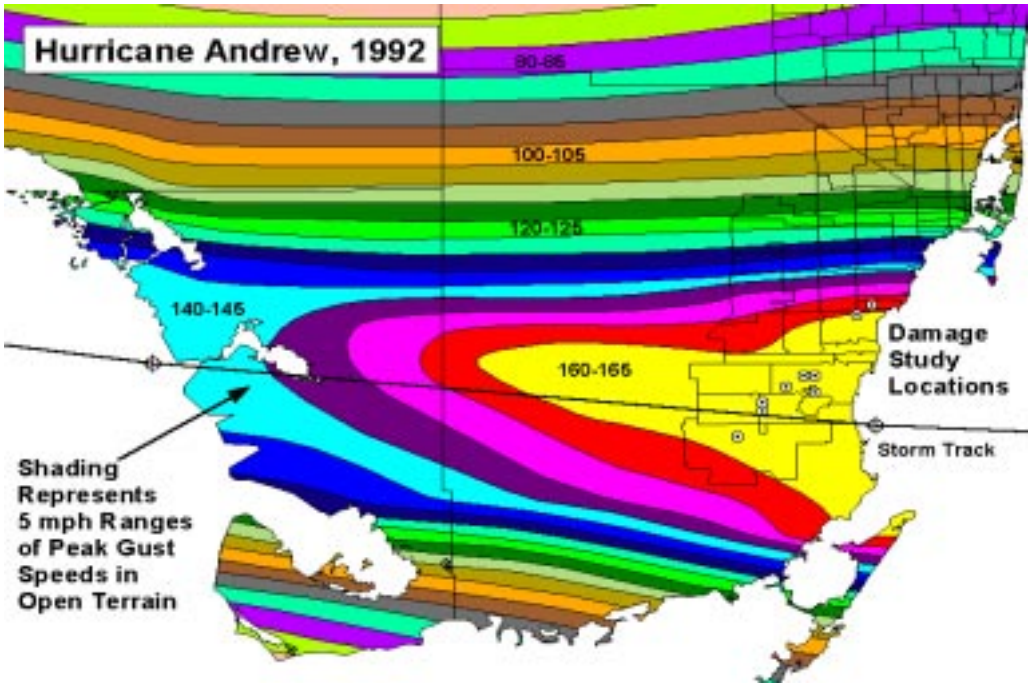
Hurricane Andrew struck a densely populated area of south Florida on August 24, 1992, with the peak recorded wind speed exceeding 175 mph (Reinhold, Vickery, and Powell, 1993). At speeds of 160 to 165 mph over a relatively large populated area, Hurricane Andrew was estimated to be about a 300-year return period event (Vickery and Twisdale, 1995; Vickery et al., 1998) (see Figure 1.8). Given the distance between the shoreline and the housing stock, most damage resulted from wind, rain, and wind-borne debris, not from the storm surge. Table 1.2 summarizes the key construction characteristics of the homes that experienced Hurricane Andrew's highest winds (as shown in Figure 1.8). Most homes were one-story structures with nominally reinforced masonry walls, wood-framed gable roofs, and composition shingle roofing.

Table 1.3 summarizes the key damage statistics for the sampled homes. As expected, the most frequent form of damage was related to windows and roofing, with 77 percent of the sampled homes suffering significant damage to roofing materials. Breakage of windows and destruction of roofing materials led to widespread and costly water damage to interiors and contents.

TABLE 1.2 ***Construction Characteristics of Sampled Single-Family Detached Homes in Hurricane Andrew***

Component	Construction Characteristics		
Number of stories	80% one	18% two	2% other
Roof construction	81% gable	13% hip	6% other
Wall construction	96% masonry	4% wood-framed	
Foundation type	100% slab		
Siding material	94% stucco		6% other
Roofing material	73% composition shingle	18% tile	9% other
Interior finish	Primarily gypsum board		

FIGURE 1.8 *Maximum Gust Wind Speeds Experienced in Hurricane Andrew*[1]



Source: Applied Research Associates, Raleigh, NC.
Note:
[1]Wind speeds are normalized to a standard 33-foot height over open terrain.

Roof sheathing was the most significant aspect of the structural damage, with 64 percent of the sampled homes losing one or more roof sheathing panels. As a result, about 24 percent of sampled homes experienced a partial or complete collapse of the roof framing system.

TABLE 1.3 *Components of Sampled Single-Family Detached Homes with "Moderate" or "High" Damage Ratings in Hurricane Andrew*

Component	Damage Frequency (percent of sampled homes)
Roof sheathing	24% (64%)[1]
Walls	2%
Foundation	0%
Roofing	77%
Interior finish (water damage)	85%

Source: Assessment of Damage to Single-Family Homes Caused by Hurricanes Andrew and Iniki (HUD, 1993).
Note:
[1]Percent in parentheses includes "low" damage rating and therefore corresponds to homes with roughly one or more sheathing panels lost. Other values indicate the percent of homes with moderate or high damage ratings only, including major component or structural failures such as partial roof collapse (i.e., 24 percent) due to excessive roof sheathing loss.

Given the magnitude of Hurricane Andrew, the structural (life-safety) performance of the predominantly masonry housing stock in south Florida was, with the prominent exception of roof sheathing attachment, entirely reasonable. While a subset of homes with wood-framed wall construction were not evaluated in a similarly rigorous fashion, anecdotal observations indicated that additional design and construction improvements, such as improved wall bracing, would be necessary to achieve acceptable performance levels for the newer styles of homes that tended to use wood framing. Indeed, the simple use of wood structural panel sheathing on all wood-framed homes may have avoided many of the more dramatic failures. Many of these problems were also exacerbated by shortcomings in code enforcement and compliance (i.e., quality). The following summarizes the major findings and conclusions from the statistical data and performance evaluation (HUD, 1993; HUD, 1998):

- While Hurricane Andrew exacted notable damage, overall residential performance was within expectation given the magnitude of the event and the minimum code-required roof sheathing attachment relative to the south Florida wind climate (i.e., a 6d nail).
- Masonry wall construction with nominal reinforcement (less than that required by current engineering specifications) and roof tie-down connections performed reasonably well and evidenced low damage frequencies, even through most homes experienced breached envelopes (i.e., broken windows).
- Failure of code-required roof tie-down straps were infrequent (i.e., less than 10 percent of the housing stock).
- Two-story homes sustained significantly (95 percent confidence level) greater damage than one-story homes.
- Hip roofs experienced significantly (95 percent confidence level) less damage than gable roofs on homes with otherwise similar characteristics.

Some key recommendations on wind-resistant design and construction include the following:

- Significant benefits in reducing the most frequent forms of hurricane damage can be attained by focusing on critical construction details related to the building envelope, such as correct spacing of roof sheathing nails (particularly at gable ends), adequate use of roof tie-downs, and window protection in the more extreme hurricane-prone environments along the southern U.S. coast.
- While construction quality was not the primary determinant of construction performance on an overall population basis, it is a significant factor that should be addressed by proper inspection of key components related to the performance of the structure, particularly connections.
- Reasonable assumptions are essential when realistically determining wind loads to ensure efficient design of wind-resistant housing.

Assumptions pertain to wind exposure condition, the internal pressure condition, and other factors as addressed later in Chapter 3.

Chapters 3 through 7 present design methods and guidance that address many of the above concerns.

Hurricane Opal

Hurricane Opal struck the Florida panhandle near Pensacola on October 4, 1995, with wind speeds between 100 and 115 mph at peak gust (normalized to an open exposure and elevation of 33 feet) over the sample region of the housing stock (Powell and Houston, 1995). Again, roofing (i.e., shingles) was the most common source of damage, occurring in 4 percent of the sampled housing stock (NAHBRC, 1996). Roof sheathing damage occurred in less than 2 percent of the affected housing stock.

The analysis of Hurricane Opal contrasts sharply with the Hurricane Andrew study. Aside from Hurricane Opal's much lower wind speeds, most homes were shielded by trees, whereas homes in south Florida were subjected to typical suburban residential exposure with relatively few trees (wind exposure B). Hurricane Andrew denuded any trees in the path of strongest wind. Clearly, housing performance in protected, noncoastal exposures is improved because of the generally less severe wind exposure and the shielding provided when trees are present. However, trees become less reliable sources of protection in more extreme hurricane-prone areas.

Northridge Earthquake

While the performance of houses in earthquakes provides objective data for measuring the acceptability of past and present seismic design and building construction practices, typical damage assessments have been based on "worst-case" observations of the most catastrophic forms of damage, leading to a skewed view of the performance of the overall population of structures. The information presented in this section is, however, based on two related studies that, like the hurricane studies, rely on objective methods to document and evaluate the overall performance of single-family attached and detached dwellings (HUD, 1994; HUD, 1999).

The Northridge Earthquake occurred at 4:31 a.m. on January 17, 1994. Estimates of the severity of the event place it at a magnitude of 6.4 on the Richter scale (Hall, 1994). Although considered a moderately strong tremor, the Northridge Earthquake produced some of the worst ground motions in recorded history for the United States, with estimated return periods of more than 10,000 years. For the most part, these extreme ground motions were highly localized and not necessarily representative of the general near-field conditions that produced ground motions representative of a 200- to 500-year return period event (HUD, 1999).

Table 1.4 summarizes the single-family detached housing characteristics documented in the survey. About 90 percent of the homes in the sample were built before the 1971 San Fernando Valley Earthquake, at which time simple prescriptive requirements were normal for single-family detached home construction. About 60 percent of the homes were built during the 1950s and 1960s, with the rest

constructed between the 1920s and early 1990s. Styles ranged from complex custom homes to simple affordable homes. All homes in the sample had wood exterior wall framing, and most did not use structural sheathing for wall bracing. Instead, wood let-in braces, Portland cement stucco, and interior wall finishes of plaster or gypsum wall board provided lateral racking resistance. Most of the crawl space foundations used full-height concrete or masonry stem walls, not wood cripple walls that are known to be prone to damage when not properly braced.

TABLE 1.4 Construction Characteristics of Sampled Single-Family Detached Dwellings

Component	Frequency of Construction Characteristics		
Number of stories	79% one	18% two	3% other
Wall sheathing	80% none	7% plywood	13% unknown
Foundation type	68% crawl space	34% slab	8% other
Exterior finish	50% stucco/mix	45% stucco only	6% other
Interior finish	60% plaster board	26% gypsum board	14% other/unknown

Source: HUD, 1994.

Table 1.5 shows the performance of the sampled single-family detached homes. Performance is represented by the percent of the total sample of homes that fell within four damage rating categories for various components of the structure (HUD, 1994).

TABLE 1.5 Damage to Sampled Single-Family Detached Homes in the Northridge Earthquake (percent of sampled homes)

Estimated Damage within Survey Area	No Damage	Low Damage	Moderate Damage	High Damage
Foundation	90.2%	8.0%	0.9%	0.9%
Walls	98.1%	1.9%	0.0%	0.0%
Roof	99.4%	0.6%	0.0%	0.0%
Exterior finish	50.7%	46.1%	2.9%	0.3%
Interior finish	49.8%	46.0%	4.2%	0.0%

Source: HUD, 1994.

Serious structural damage to foundations, wall framing, and roof framing was limited to a small proportion of the surveyed homes. In general, the homes suffered minimal damage to the elements that are critical to occupant safety. Of the structural elements, damage was most common in foundation systems. The small percent of surveyed homes (about 2 percent) that experienced moderate to high foundation damage were located in areas that endured localized ground effects (i.e., fissuring or liquefaction) or problems associated with steep hillside sites.

Interior and exterior finishes suffered more widespread damage, with only about half the residences escaping unscathed. However, most of the interior/exterior finish damage in single-family detached homes was limited to the lowest rating categories. Damage to stucco usually appeared as hairline cracks radiating from the corners of openings—particularly larger openings such as garage doors—or along the tops of foundations. Interior finish damage paralleled the occurrence of exterior

finish (stucco) damage. Resilient finishes—such as wood panel or lap board siding—fared well and often showed no evidence of damage even when stucco on other areas of the same unit was moderately damaged. However, these seemingly minor types of damage were undoubtedly a major source of the economic impact in terms of insurance claims and repair cost. In addition, it is often difficult to separate the damage into categories of "structural" and "nonstructural," particularly when some systems, such as Portland cement stucco, are used as an exterior cladding as well as structural bracing. It is also important to recognize that the Northridge Earthquake is not considered a "maximum" earthquake event.

The key findings of an evaluation of the above performance data are summarized below (HUD, 1999). Overall, the damage relative to key design features showed no discernable pattern, implying great uncertainties in seismic design and building performance that may not be effectively addressed by simply making buildings "stronger."

The amount of wall bracing using conventional stucco and let-in braces typically ranged from 30 to 60 percent of the wall length (based on the street-facing walls of the sampled one-story homes). However, there was no observable or statistically significant trend between amount of damage and amount of stucco wall bracing. Since current seismic design theory implies that more bracing is better, the Northridge findings are fundamentally challenging yet offer little in the way of a better design theory. At best, the result may be explained by the fact that numerous factors govern the performance of a particular building in a major seismic event. For example, conventional seismic design, while intending to do so, may not effectively consider the optimization of flexibility, ductility, dampening, and strength–all of which are seemingly important.

The horizontal ground motions experienced over the sample region for the study ranged from 0.26 to 2.7 g for the short-period (0.2 second) spectral response acceleration and from 0.10 to 1.17 g for the long-period (1 second) spectral response acceleration. The near-field ground motions represent a range between the 100- and 14,000-year return period, but a 200- to 500-year return period is more representative of the general ground motion experienced. The short-period ground motion (typically used in the design of light-frame structures) had no apparent correlation with the amount of damage observed in the sampled homes, although a slight trend with respect to the long-period ground motion was observed in the data.

The Northridge damage survey and evaluation of statistical data suggest the following conclusions and recommendations (HUD, 1994; HUD, 1999):

- Severe structural damage to single-family detached homes was infrequent and primarily limited to foundation systems. Less than 2 percent of single-family detached homes suffered moderate to high levels of foundation damage, and most occurrences were associated with localized site conditions, including liquefaction, fissuring, and steep hillsides.
- Structural damage to wall and roof framing in single-family detached homes was limited to low levels for about 2 percent of the walls and for less than 1 percent of all roofs.
- Exterior stucco and interior finishes experienced the most widespread damage, with 50 percent of all single-family detached homes suffering at

least minor damage and roughly 4 percent of homes sustaining moderate to high damage. Common finish damage was related to stucco and drywall/plaster cracks emanating from the foundation or wall openings.

- Homes on slab foundations suffered some degree of damage to exterior stucco finishes in about 30 percent of the sample; crawl space homes approached a 60 percent stucco damage rate that was commonly associated with the flexibility of the wall-floor-foundation interface.
- Peak ground motion records in the near-field did not prove to be a significant factor in relation to the level of damage as indicated by the occurrence of stucco cracking. Peak ground acceleration may not of itself be a reliable design parameter in relation to the seismic performance of light-frame homes. Similarly, the amount of stucco wall bracing on street-facing walls showed a negligible relationship with the variable amount of damage experienced in the sampled housing.

Some basic design recommendations call for

- simplifying seismic design requirements to a degree commensurate with knowledge and uncertainty regarding how homes actually perform (see Chapter 3);
- using fully sheathed construction in high-hazard seismic regions (see Chapter 6);
- taking design precautions or avoiding steeply sloped sites or sites with weak soils; and,
- when possible, avoiding brittle interior and exterior wall finish systems in high-hazard seismic regions.

1.7 Summary

Housing in the United States has evolved over time under the influence of a variety of factors. While available resources and the economy continue to play a significant role, building codes, consumer preferences, and alternative construction materials are becoming increasingly important factors. In particular, many local building codes in the United States now require homes to be specially designed rather than following conventional construction practices. In part, this apparent trend may be attributed to changing perceptions regarding housing performance in high-risk areas. Therefore, greater emphasis must be placed on efficient structural design of housing. While efficient design should also strive to improve construction quality through simplified construction, it also places greater importance on the quality of installation required to achieve the intended performance without otherwise relying on “overdesign” to compensate partially for real or perceived problems in installation quality.

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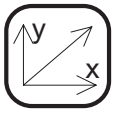


CHAPTER 2

Structural Design Concepts

2.1 General

This chapter reviews some fundamental concepts of structural design and presents them in a manner relevant to the design of light-frame residential structures. The concepts form the basis for understanding the design procedures and overall design approach addressed in the remaining chapters of the guide. With this conceptual background, it is hoped that the designer will gain a greater appreciation for creative and efficient design of homes, particularly the many assumptions that must be made.

2.2 What Is Structural Design?

The process of structural design is simple in concept but complex in detail. It involves the analysis of a proposed structure to show that its resistance or strength will meet or exceed a reasonable expectation. This expectation is usually expressed by a specified load or demand and an acceptable margin of safety that constitutes a performance goal for a structure.

The performance goals of structural design are multifaceted. Foremost, a structure must perform its intended function safely over its useful life. Safety is discussed later in this chapter. The concept of useful life implies considerations of durability and establishes the basis for considering the cumulative exposure to time-varying risks (i.e., corrosive environments, occupant loads, snow loads, wind loads, and seismic loads). Given, however, that performance is inextricably linked to cost, owners, builders, and designers must consider economic limits to the primary goals of safety and durability.

The appropriate balance between the two competing considerations of performance and cost is a discipline that guides the “art” of determining value in building design and construction. However, value is judged by the “eye of the

beholder," and what is an acceptable value to one person may not be acceptable value to another (i.e., too costly versus not safe enough or not important versus important). For this reason, political processes mediate minimum goals for building design and structural performance, with minimum value decisions embodied in building codes and engineering standards that are adopted as law.

In view of the above discussion, a structural designer may appear to have little control over the fundamental goals of structural design, except to comply with or exceed the minimum limits established by law. While this is generally true, a designer can still do much to optimize a design through alternative means and methods that call for more efficient analysis techniques, creative design detailing, and the use of innovative construction materials and methods.

In summary, the goals of structural design are generally defined by law and reflect the collective interpretation of general public welfare by those involved in the development and local adoption of building codes. The designer's role is to meet the goals of structural design as efficiently as possible and to satisfy a client's objectives within the intent of the building code. Designers must bring to bear the fullest extent of their abilities, including creativity, knowledge, experience, judgment, ethics, and communication–aspects of design that are within the control of the individual designer and integral to a comprehensive approach to design. Structural design is much, much more than simply crunching numbers.

2.3 Load Conditions and Structural System Response

The concepts presented in this section provide an overview of building loads and their effect on the structural response of typical wood-framed homes. As shown in Table 2.1, building loads can be divided into two types based on the orientation of the structural actions or forces that they induce: vertical loads and horizontal (i.e., lateral) loads.

TABLE 2.1 ***Building Loads Categorized by Orientation***

Vertical Loads	Horizontal (Lateral) Loads
• Dead (gravity) • Live (gravity) • Snow (gravity) • Wind (uplift on roof) • Seismic and wind (overturning) • Seismic (vertical ground motion)	• Wind • Seismic (horizontal ground motion) • Flood (static and dynamic hydraulic forces) • Soil (active lateral pressure)

2.3.1 Vertical Loads

Gravity loads act in the same direction as gravity (i.e., downward or vertically) and include dead, live, and snow loads. They are generally static in nature and usually considered a uniformly distributed or concentrated load. Thus, determining a gravity load on a beam or column is a relatively simple exercise that uses the concept of tributary areas to assign loads to structural elements. The tributary area is the area of the building construction that is supported by a structural element, including the dead load (i.e., weight of the construction) and any applied loads (i.e., live load). For example, the tributary gravity load on a floor joist would include the uniform floor load (dead and live) applied to the area of floor supported by the individual joist. The structural designer then selects a standard beam or column model to analyze bearing connection forces (i.e., reactions), internal stresses (i.e., bending stresses, shear stresses, and axial stresses), and stability of the structural member or system; refer to Appendix A for beam equations. The selection of an appropriate analytic model is, however, no trivial matter, especially if the structural system departs significantly from traditional engineering assumptions that are based on rigid body and elastic behavior. Such departures from traditional assumptions are particularly relevant to the structural systems that comprise many parts of a house, but to varying degrees.

Wind uplift forces are generated by negative (suction) pressures acting in an outward direction from the surface of the roof in response to the aerodynamics of wind flowing over and around the building. As with gravity loads, the influence of wind uplift pressures on a structure or assembly (i.e., roof) are analyzed by using the concept of tributary areas and uniformly distributed loads. The major difference is that wind pressures act perpendicular to the building surface (not in the direction of gravity) and that pressures vary according to the size of the tributary area and its location on the building, particularly proximity to changes in geometry (e.g., eaves, corners, and ridges). Even though the wind loads are dynamic and highly variable, the design approach is based on a maximum static load (i.e., pressure) equivalent.

Vertical forces are also created by overturning reactions due to wind and seismic lateral loads acting on the overall building and its lateral force resisting systems. Earthquakes also produce vertical ground motions or accelerations which increase the effect of gravity loads. However, vertical earthquake loads are usually considered to be implicitly addressed in the gravity load analysis of a light-frame building.

2.3.2 Lateral Loads

The primary loads that produce *lateral forces* on buildings are attributable to forces associated with wind, seismic ground motion, floods, and soil. Wind and seismic lateral loads apply to the entire building. Lateral forces from wind are generated by positive wind pressures on the windward face of the building and by negative pressures on the leeward face of the building, creating a combined push-and-pull effect. Seismic lateral forces are generated by a structure's dynamic inertial response to cyclic ground movement. The magnitude of the seismic shear

(i.e., lateral) load depends on the magnitude of the ground motion, the building's mass, and the dynamic structural response characteristics (i.e., dampening, ductility, natural period of vibration, etc.). For houses and other similar low-rise structures, a simplified seismic load analysis employs equivalent static forces based on fundamental Newtonian mechanics (F=ma) with somewhat subjective (i.e., experience-based) adjustments to account for inelastic, ductile response characteristics of various building systems. Flood loads are generally minimized by elevating the structure on a properly designed foundation or avoided by not building in a flood plain. Lateral loads from moving flood waters and static hydraulic pressure are substantial. Soil lateral loads apply specifically to foundation wall design, mainly as an "out-of-plane" bending load on the wall.

Lateral loads also produce an *overturning* moment that must be offset by the dead load and connections of the building. Therefore, overturning forces on connections designed to restrain components from rotating or the building from overturning must be considered. Since wind is capable of generating simultaneous roof uplift and lateral loads, the uplift component of the wind load exacerbates the overturning tension forces due to the lateral component of the wind load. Conversely, the dead load may be sufficient to offset the overturning and uplift forces as is often the case in lower design wind conditions and in many seismic design conditions.

2.3.3 Structural Systems

As far back as 1948, it was determined that "conventions in general use for wood, steel and concrete structures are not very helpful for designing houses because few are applicable" (NBS, 1948). More specifically, the NBS document encourages the use of more advanced methods of structural analysis for homes. Unfortunately, the study in question and all subsequent studies addressing the topic of system performance in housing have not led to the development or application of any significant improvement in the codified design practice as applied to housing systems. This lack of application is partly due to the conservative nature of the engineering process and partly due to the difficulty of translating the results of narrowly-focused structural systems studies to general design applications. Since this document is narrowly scoped to address residential construction, relevant system-based studies and design information for housing are discussed, referenced, and applied as appropriate.

If a structural member is part of a *system*, as is typically the case in light-frame residential construction, its response is altered by the strength and stiffness characteristics of the system as a whole. In general, system performance includes two basic concepts known as *load sharing* and *composite action*. Load sharing is found in repetitive member systems (i.e., wood framing) and reflects the ability of the load on one member to be shared by another or, in the case of a uniform load, the ability of some of the load on a weaker member to be carried by adjacent members. Composite action is found in assemblies of components that, when connected to one another, form a "composite member" with greater capacity and stiffness than the sum of the component parts. However, the amount of composite action in a system depends on the manner in which the various system elements are connected. The aim is to achieve a higher effective section modulus than the

component members taken separately. For example, when floor sheathing is nailed and glued to floor joists, the floor system realizes a greater degree of composite action than a floor with sheathing that is merely nailed; the adhesive between components helps prevent shear slippage, particularly if a rigid adhesive is used. Slippage due to shear stresses transferred between the component parts necessitates consideration of partial composite action, which depends on the stiffness of an assembly's connections. Therefore, consideration of the floor as a system of fully composite T-beams may lead to an unconservative solution whereas the typical approach of only considering the floor joist member without composite system effect will lead to a conservative design.

This guide addresses the strength-enhancing effect of load sharing and partial composite action when information is available for practical design guidance. Establishment of repetitive-member increase factors (also called system factors) for general design use is a difficult task because the amount of system effect can vary substantially depending on system assembly and materials. Therefore, system factors for general design use are necessarily conservative to cover broad conditions. Those that more accurately depict system effects also require a more exact description of and compliance with specific assembly details and material specifications.

It should be recognized, however, that system effects do not only affect the strength and stiffness of light-frame assemblies (including walls, floors, and roofs). They also alter the classical understanding of how loads are transferred among the various assemblies of a complex structural system, including a complete wood-framed home. For example, floor joists are sometimes doubled under nonload-bearing partition walls "because of the added dead load and resulting stresses" determined in accordance with accepted engineering practice. Such practice is based on a conservative assumption regarding the load path and the structural response. That is, the partition wall does create an additional load, but the partition wall is relatively rigid and actually acts as a deep beam, particularly when the top and bottom are attached to the ceiling and floor framing, respectively. As the floor is loaded and deflects, the interior wall helps resist the load. Of course, the magnitude of effect depends on the wall configuration (i.e., amount of openings) and other factors.

The above example of composite action due to the interaction of separate structural systems or subassemblies points to the improved structural response of the floor system such that it is able to carry more dead and live load than if the partition wall were absent. One whole-house assembly test has demonstrated this effect (Hurst, 1965). Hence, a double joist should not be required under a typical nonload-bearing partition; in fact, a single joist may not even be required directly below the partition, assuming that the floor sheathing is adequately specified to support the partition between the joists. While this condition cannot yet be duplicated in a standard analytic form conducive to simple engineering analysis, a designer should be aware of the concept when making design assumptions regarding light-frame residential construction.

At this point, the reader should consider that the response of a structural system, not just its individual elements, determines the manner in which a structure distributes and resists horizontal and vertical loads. For wood-framed systems, the departure from calculations based on classical engineering mechanics

(i.e., single members with standard tributary areas and assumed elastic behavior) and simplistic assumptions regarding load path can be substantial.

2.4 Load Path

Loads produce stresses on various systems, members, and connections as load-induced forces are transferred down through the structure to the ground. The path through which loads are transferred is known as the load path. A continuous load path is capable of resisting and transferring the loads that are realized throughout the structure from the point of load origination down to the foundation.

As noted, the load path in a conventional home may be extremely complex because of the structural configuration and system effects that can result in substantial load sharing, partial composite action, and a redistribution of forces that depart from traditional engineering concepts. In fact, such complexity is an advantage that often goes overlooked in typical engineering analyses.

Further, because interior nonload-bearing partitions are usually ignored in a structural analysis, the actual load distribution is likely to be markedly different from that assumed in an elementary structural analysis. However, a strict accounting of structural effects would require analytic methods that are not yet available for general use. Even if it were possible to capture the full structural effects, future alterations to the building interior could effectively change the system upon which the design was based. Thus, there are practical and technical limits to the consideration of system effects and their relationships to the load path in homes.

2.4.1 The Vertical Load Path

Figures 2.1 and 2.2 illustrate vertically oriented loads created, respectively, by gravity and wind uplift. It should be noted that the wind uplift load originates on the roof from suction forces that act perpendicular to the exterior surface of the roof as well as from internal pressure acting perpendicular to the interior surface of the roof-ceiling assembly in an outward direction. In addition, overturning forces resulting from lateral wind or seismic forces create vertical uplift loads (not shown in Figure 2.2). In fact, a separate analysis of the lateral load path usually addresses overturning forces, necessitating separate overturning connections for buildings located in high-hazard wind or seismic areas (see Section 2.3). As addressed in Chapter 6, it may be feasible to combine these vertical forces and design a simple load path to accommodate wind uplift and overturning forces simultaneously.

FIGURE 2.1 Illustration of the Vertical Load Path for Gravity Loads

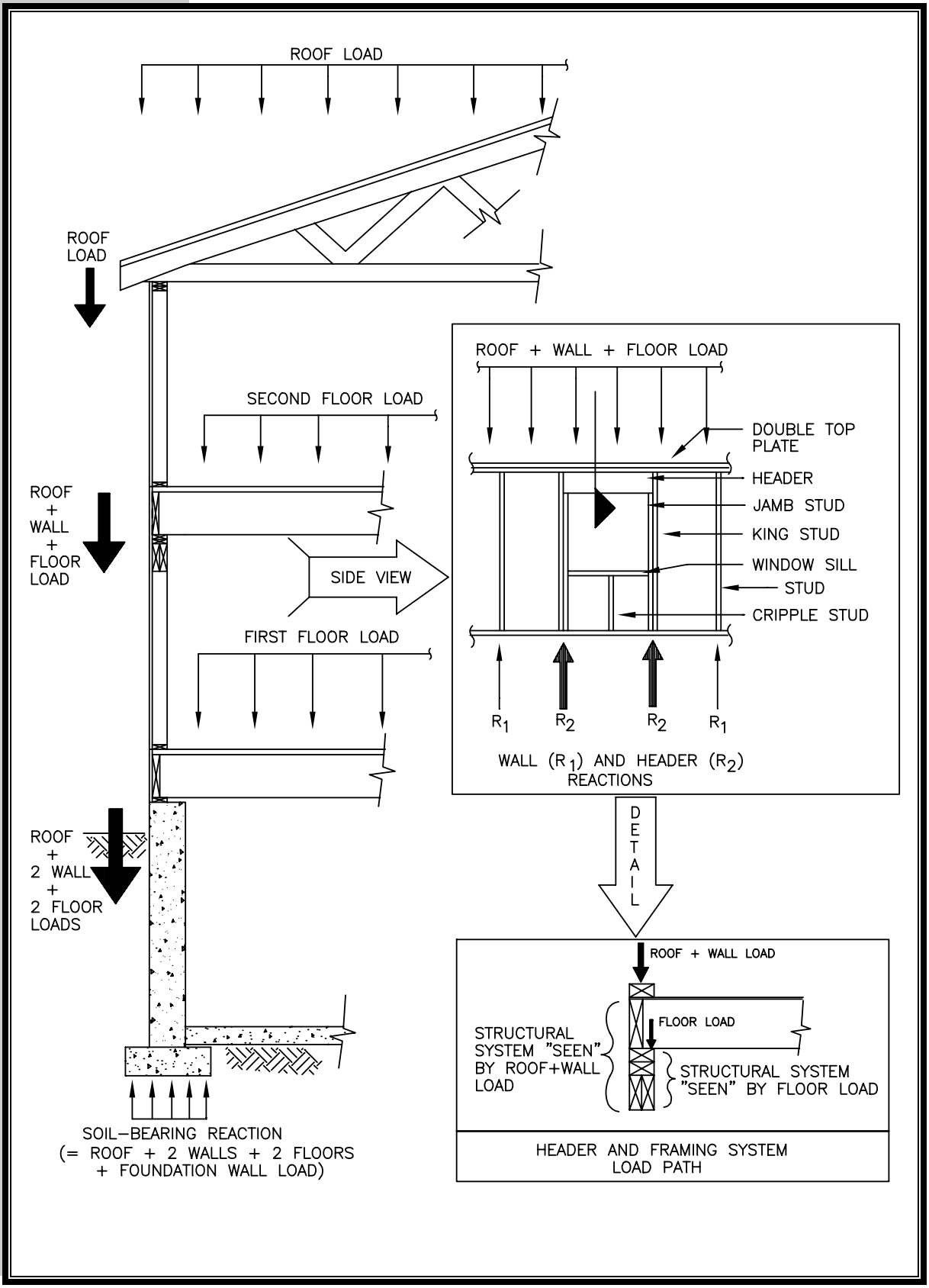
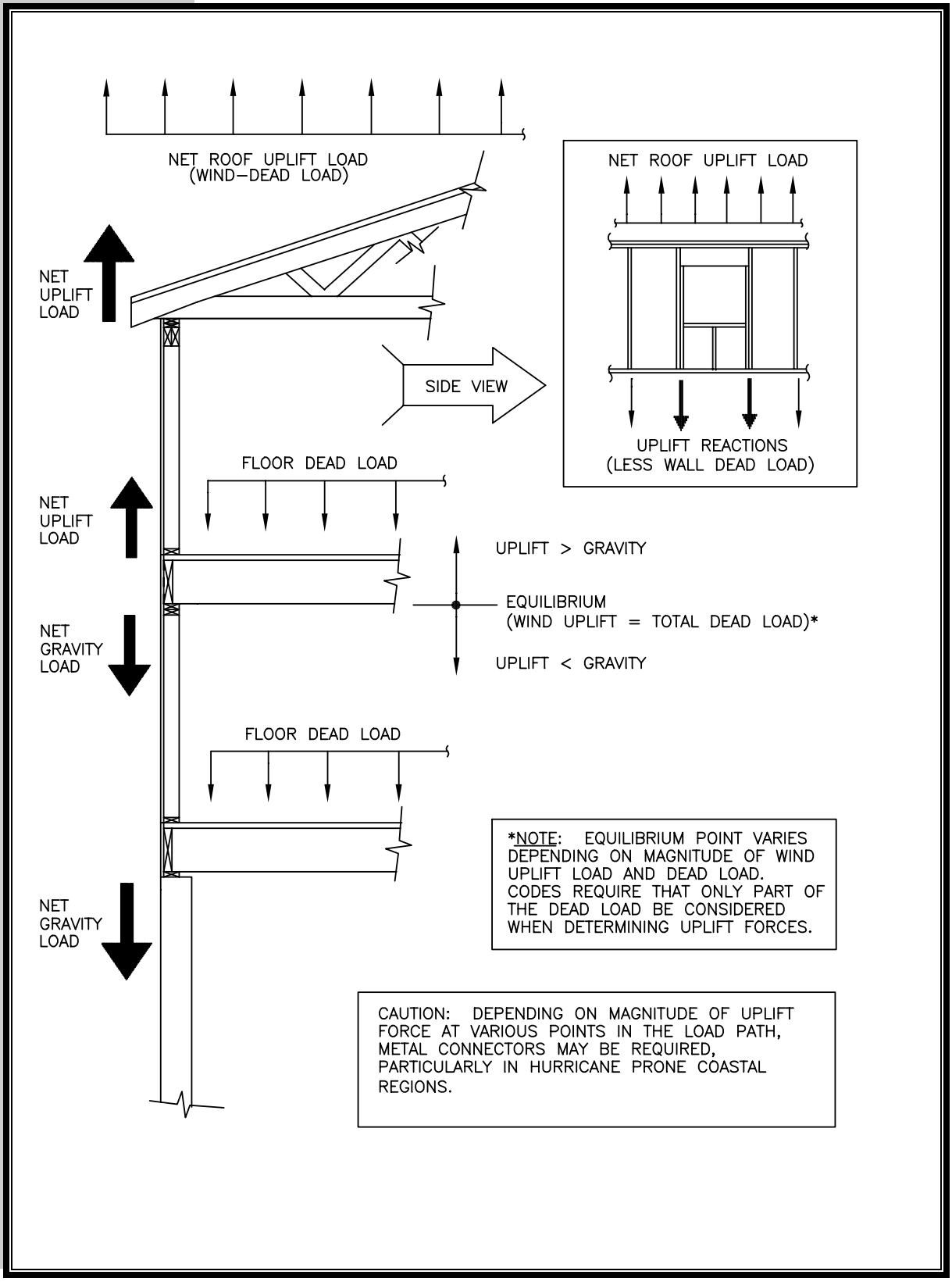


FIGURE 2.2 Illustration of the Vertical Load Path for Wind Uplift



In a typical two-story home, the load path for gravity loads and wind uplift involves the following structural elements:

- roof sheathing;
- roof sheathing attachment;
- roof framing member (rafter or truss);
- roof-to-wall connection;
- second-story wall components (top plate, studs, sole plate, headers, wall sheathing, and their interconnections);
- second-story-wall-to-second-floor connection;
- second-floor-to-first-story-wall connection;
- first-story wall components (same as second story);
- first-story-wall-to-first-floor or foundation connection;
- first-floor-to-foundation connection; and
- foundation construction.

From the above list, it is obvious that there are numerous members, assemblies, and connections to consider in tracking the gravity and wind uplift load paths in a typical wood-framed home. The load path itself is complex, even for elements such as headers that are generally considered simple beams. Usually, the header is part of a structural system (see Figure 2.1), not an individual element single-handedly resisting the entire load originating from above. Thus, a framing system around a wall opening, not just a header, comprises a load path.

Figure 2.1 also demonstrates the need for appropriately considering the combination of loads as the load moves "down" the load path. Elements that experience loads from multiple sources (e.g., the roof and one or more floors) can be significantly overdesigned if design loads are not proportioned or reduced to account for the improbability that all loads will occur at the same time. Of course, the dead load is always present, but the live loads are transient; even when one floor load is at its life-time maximum, it is likely that the others will be at only a fraction of their design load. Current design load standards generally allow for multiple transient load reductions. However, with multiple transient load reduction factors intended for general use, they may not effectively address conditions relevant to a specific type of construction (i.e., residential).

Consider the soil-bearing reaction at the bottom of the footing in Figure 2.1. As implied by the illustration, the soil-bearing force is equivalent to the sum of all tributary loads–dead and live. However, it is important to understand the combined load in the context of design loads. Floor design live loads are based on a life-time maximum estimate for a single floor in a single level of a building. But, in the case of homes, the upper and lower stories or occupancy conditions typically differ. When one load is at its maximum, the other is likely to be at a fraction of its maximum. Yet, designers are not able to consider the live loads of the two floors as separate transient loads because specific guidance is not currently available. In concept, the combined live load should therefore be reduced by an appropriate factor, or one of the loads should be set at a point-in-time value that is a fraction of its design live load. For residential construction, the floor design live load is either 30 psf (for bedroom areas) or 40 psf (for other areas), although some codes require a design floor live load of 40 psf for all areas.

In contrast, average sustained live loads during typical use conditions are about 6 psf (with one standard deviation of 3 psf), which is about 15 to 20 percent of the design live load (Chalk and Corotis, 1980). If actual loading conditions are not rationally considered in a design, the result may be excessive footing widths, header sizes, and so forth.

When tracking the wind uplift load path (Figure 2.2), the designer must consider the offsetting effect of the dead load as it increases down the load path. However, it should be noted that building codes and design standards do not permit the consideration of any part of the sustained live load in offsetting wind uplift, even though it is highly probable that some minimum point-in-time value of floor live load is present if the building is in use, i.e., furnished and/or occupied. In addition, other "nonengineered" load paths, such as provided by interior walls and partitions, are not typically considered. While these are prudent limits, they help explain why certain structures may not "calculate" but otherwise perform adequately.

Depending on the code, it is also common to consider only two-thirds of the dead load when analyzing a structure's net wind uplift forces. The two-thirds provision is a way of preventing the potential error of requiring insufficient connections where a zero uplift value is calculated in accordance with a nominal design wind load (as opposed to the ultimate wind event that is implied by the use of a safety margin for material strength in unison with a nominal design wind speed). Furthermore, code developers have expressed a concern that engineers might overestimate actual dead loads.

For complicated house configurations, a load of any type may vary considerably at different points in the structure, necessitating a decision of whether to design for the worst case or to accommodate the variations. Often the worst-case condition is applied to the entire structure even when only a limited part of the structure is affected. For example, a floor joist or header may be sized for the worst-case span and used throughout the structure. The worst-case decision is justified only when the benefit of a more intensive design effort is not offset by a significant cost reduction. It is also important to be mindful of the greater construction complexity that usually results from a more detailed analysis of various design conditions. Simplification and cost reduction are both important design objectives, but they may often be mutually exclusive. However, the consideration of system effects in design, as discussed earlier, may result in both simplification and cost efficiencies that improve the quality of the finished product.

One helpful attribute of traditional platform-framed home construction is that the floor and roof gravity loads are typically transferred through bearing points, not connections. Thus, connections may contribute little to the structural performance of homes with respect to vertical loads associated with gravity (i.e., dead, live, and snow loads). While outdoor deck collapses have occurred on occasion, the failure in most instances is associated with an inadequate or deteriorated connection to the house, not a bearing connection.

By contrast, metal plate-connected roof and floor trusses rely on connections to resist gravity loads, but these engineered components are designed and produced in accordance with a proven standard and are generally highly reliable (TPI, 1996). Indeed, the metal plate-connected wood truss was first conceived in Florida in the 1950s to respond to the need for improved roof

structural performance, particularly with respect to connections in roof construction (WTCA, 1998).

In high-wind climates where the design wind uplift load approaches the offsetting dead load, the consideration of connection design in wood-framed assemblies becomes critical for roofs, walls, and floors. In fact, the importance of connections in conventionally built homes is evidenced by the common loss of weakly attached roof sheathing or roofs in extreme wind events such as moderate- to large-magnitude hurricanes.

Newer prescriptive code provisions have addressed many of the historic structural wind damage problems by specifying more stringent general requirements (SBCCI, 1999; AF&PA, 1996). In many cases, the newer high-wind prescriptive construction requirements may be improved by more efficient site-specific design solutions that consider wind exposure, system effects, and other analytic improvements. The same can be said for prescriptive seismic provisions found in the latest building codes for conventional residential construction (ICC, 1999; ICBO, 1997).

2.4.2 Lateral Load Path

The overall system that provides lateral resistance and stability to a building is known as the lateral force resisting system (LFRS). In light-frame construction, the LFRS includes shear walls and horizontal diaphragms. Shear walls are walls that are typically braced or clad with structural sheathing panels to resist racking forces. Horizontal diaphragms are floor and roof assemblies that are also usually clad with structural sheathing panels. Though more complicated and difficult to visualize, the lateral forces imposed on a building from wind or seismic action also follow a load path that distributes and transfers shear and overturning forces from lateral loads. The lateral loads of primary interest are those resulting from

- the horizontal component of wind pressures on the building's exterior surface area; and
- the inertial response of a building's mass and structural system to seismic ground motions.

As seen in Figure 2.3, the lateral load path in wood-framed construction involves entire structural assemblies (i.e., walls, floors, and roofs) and their interconnections, not just individual elements or frames as would be the case with typical steel or concrete buildings that use discrete braced framing systems. The distribution of loads in Figure 2.3's three-dimensional load path depends on the relative stiffness of the various components, connections, and assemblies that comprise the LFRS. To complicate the problem further, stiffness is difficult to determine due to the nonlinearity of the load-displacement characteristics of wood-framed assemblies and their interconnections. Figure 2.4 illustrates a deformed light-frame building under lateral load; the deformations are exaggerated for conceptual purposes.

FIGURE 2.3 Illustration of the Lateral Load Path

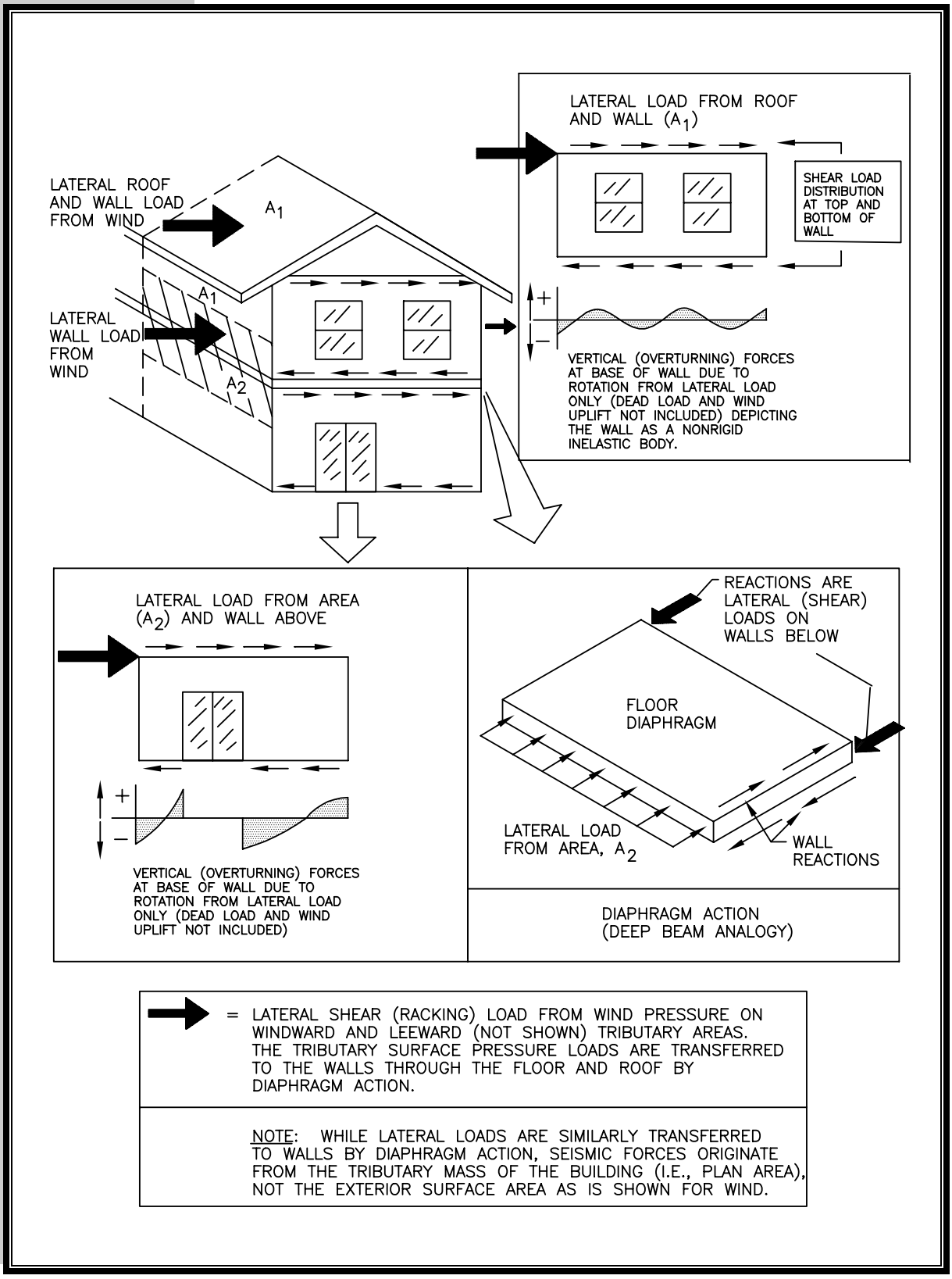
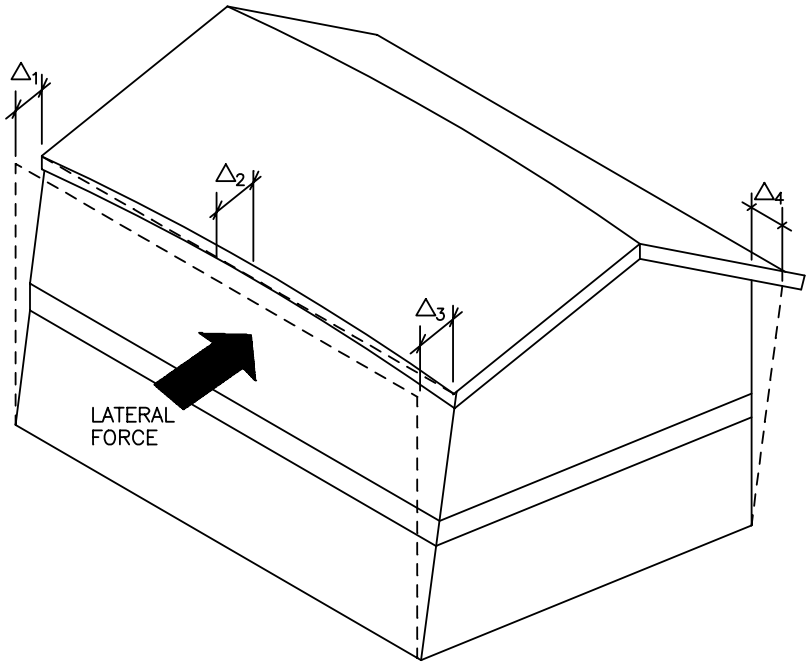


FIGURE 2.4 Illustration of Building Deformation under Lateral Load



NOTE: IF STIFFNESS OR LOAD IS NONSYMMETRICAL, BUILDING ROTATION OCCURS ($\Delta_1 \neq \Delta_3$) AND LOADS ARE DISTRIBUTED BY TORSION ($\Delta_4 \neq 0$) AS WELL AS BY DIRECT SHEAR IN THE DIRECTION OF THE LATERAL FORCE. THIS CONDITION VARIES BUT IS A REALITY FOR MOST DESIGNS. Δ_2 IS THE BENDING DEFORMATION OF THE HORIZONTAL DIAPHRAGM (I.E., ROOF).

Lateral forces from wind and seismic loads also create overturning forces that cause a "tipping" or "roll-over" effect. When these forces are resisted, a building is prevented from overturning in the direction of the lateral load. On a smaller scale than the whole building, overturning forces are realized at the shear walls of the LFRS such that the shear walls must be restrained from rotating or rocking on their base by proper connection. On an even smaller scale, the forces are realized in the individual shear wall segments between openings in the walls. As shown in Figure 2.3, the overturning forces are not necessarily distributed as might be predicted. The magnitude and distribution of the overturning force can depart significantly from a typical engineering analysis depending on the building or wall configuration.

The overturning force diagrams in Figure 2.3 are based on conventionally built homes constructed without hold-down devices positioned to restrain shear wall segments independently. It should be noted that the effect of dead loads that may offset the overturning force and of wind uplift loads that may increase the overturning force is not necessarily depicted in Figure 2.3's conceptual plots of overturning forces at the base of the walls. If rigid steel hold-down devices are used in designing the LFRS, the wall begins to behave in a manner similar to a rigid body at the level of individual shear wall segments, particularly when the wall is broken into discrete segments as a result of the configuration of openings in a wall line.

In summary, significant judgment and uncertainty attend the design process for determining building loads and resistance, including definition of the load path and the selection of suitable analytic methods. Designers are often compelled to comply with somewhat arbitrary design provisions or engineering conventions, even when such conventions are questionable or incomplete for particular applications such as a wood-framed home. At the same time, individual designers are not always equipped with sufficient technical information or experience to depart from traditional design conventions. Therefore, this guide is intended to serve as a resource for designers who are considering the use of improved analytic methods when current analytic approaches may be lacking.

2.5 Structural Safety

Before addressing the "nuts and bolts" of structural design of single-family dwellings, it is important to understand the fundamental concept of safety. While safety is generally based on rational principles of risk and probability theory, it is also subject to judgment, particularly the experience and understanding of those who participate in the development of building codes and design standards. For this reason, it is not uncommon to find differences in various code-approved sources for design loads, load combinations, load factors, and other features that affect structural safety and design economy. Despite these inconsistencies, the aim of any design approach is to ensure that the probability of failure (i.e., load exceeding resistance) is acceptably small or, conversely, that the level of safety is sufficiently high.

A common misconception holds that design loads determine the amount of "safety" achieved. It is for this reason that some people tend to focus on design loads to solve real or perceived problems associated with structural performance

(i.e., safety or property damage). For example, a typical conclusion reached in the aftermath of Hurricane Andrew was that the storm's wind speed exceeded the design wind speed map value; therefore, the wind map (i.e., design load) was insufficient. In other cases, such as the Northridge Earthquake, reaction to various anecdotal observations resulted in increased safety factors for certain materials (i.e., wood design values were decreased by 25 percent by the City of Los Angeles, California). In reality, several factors affect the level of safety just as several factors determine the level of performance realized by buildings in a single extreme event such as Hurricane Andrew or the Northridge Earthquake (see Chapter 1).

Structural safety is a multifaceted performance goal that integrates all objective and subjective aspects of the design process, including the following major variables:

- determination of characteristic material or assembly strength values based on tested material properties and their variabilities;

- application of a nominal or design load based on a statistical representation of load data and the data's uncertainty or variability;

- consideration of various uncertainties associated with the design practice (e.g., competency of designers and accuracy of analytic approaches), the construction practice (e.g., quality or workmanship), and durability; and

- selection of a level of safety that considers the above factors and the consequences of exceeding a specified design limit state (i.e., collapse, deformation, or the onset of "unacceptable" damage).

When the above variables are known or logically conceived, there are many ways to achieve a specified level of safety. However, as a practical necessity, the design process has been standardized to provide a reasonably consistent basis for applying the following key elements of the design process:

- characterizing strength properties for various material types (e.g., steel, wood, concrete, masonry, etc.);

- defining nominal design loads and load combinations for crucial inputs into the design process; and

- conveying an acceptable level of safety (i.e., safety margin) that can be easily and consistently applied by designers.

Institutionalized design procedures provide a basis for selecting from the vast array of structural material options available in the construction market. However, the generalizations necessary to address the multitude of design conditions rely on a simplified and standardized format and thus often overlook special aspects of a particular design application.

While the following sections discuss safety, they are intentionally basic and focus on providing the reader with a conceptual understanding of safety and probability as a fundamental aspect of engineering. Probability concepts are fundamental to modern design formats, such as load and resistance factor design (LRFD), which is also known as reliability-based design or simply strength design. The same concepts are also crucial to understanding the implications of the simple safety factor in traditional allowable stress design (ASD). As with many aspects of engineering, it is important to realize that the treatment of safety is not an exact science but rather depends on the application of sound judgment as much as on the application of complex or sophisticated statistical theories to analyze the many variables in the design process that affect reliability (Gromala et al., 1999). The following references are recommended for further study:

- *Probability Concepts in Engineering Planning and Design*, Volume I–Basic Principles (Ang and Tang, 1975)
- *CRC Structural Engineering Handbook, Chapter 29: Structural Reliability* (Chen, 1997)
- *Probabilistic Structural Mechanics Handbook: Theory and Industrial Applications* (Sundararajan, 1995)
- *Uncertainty Analysis, Loads, and Safety in Structural Engineering* (Hart, 1982)
- *Statistical Models in Engineering* (Hahn and Shapiro, 1967)

2.5.1 Nominal Design Loads

Nominal design loads are generally specified on the basis of probability, with the interchangeable terms “return period” and “mean recurrence interval” often used to describe the probability of loads. Either term represents a condition that is predicted to be met or exceeded once on average during the reference time period. For design purposes, loads are generally evaluated in terms of annual extremes (i.e., variability of the largest load experienced in any given one-year period) or maximum life-time values.

The choice of the return period used to define a nominal design load is somewhat arbitrary and must be applied appropriately in the design process. The historical use of safety factors in allowable stress design (ASD) has generally been based on a 50-year return period design load. With the advent of load and resistance factor design (LRFD), the calculation of nominal loads has shifted away from ASD for some load types. For example, earthquake design loads are now based on a 475-year return period event. As a result, a load factor of less than one (i.e., 0.7) must now be used to adjust the earthquake load basis roughly back to a 50-year return period magnitude so that the appropriate level of safety is achieved relative to allowable material strength values used in ASD. This condition is reflected in the design load combinations in Chapter 3.

The method of determining a design load also differs according to the type of load and the availability of data to evaluate the time-varying nature of loads. The derivation of various nominal loads may be assembled from information and references contained in the ASCE 7 standard (ASCE, 1999). A brief summary is provided here. Design wind loads are based on a probabilistic analysis of wind speed data collected from numerous weather stations across the United States. Given, however, the absence of sufficiently long-term weather data to quantify hurricane risk accurately, wind loads along the hurricane coastline are determined by using a hurricane simulation model that is based on past hurricane tracking records as well as on an examination of the physical characteristics of hurricanes.[1] Snow loads are based on snowfall or ground snow depth data and are correlated to roof snow loads through somewhat limited studies. Snow drift loads are conservatively based on drifting on failed roofs and therefore do not necessarily represent the snow-drifting probability that occurs at random in the building population. Earthquake loads are defined from historical ground motion data and conceptualized risk models based on direct or indirect evidence of past earthquake activity. Thus, considerable uncertainty exists in the estimation of seismic hazards, particularly in areas that are believed to have low seismicity (i.e., few events) but the potential for major seismic events. Floor live loads are modeled by using live load surveys of "point-in-time" loading conditions and hypotheses or judgment concerning extreme or maximum life-time loads. In some cases, expert panels decide on appropriate loads or related load characteristics when adequate data are not available.

In summary, the determination of load characteristics is based on historical data, risk modeling, and expert opinion, which, in turn, guide the specification of nominal design loads for general design purposes in both the ASD and LRFD formats. As noted, nominal design loads were usually based on a 50-year return period. Today, however, the calculation of seismic loads and wind loads along the hurricane coastline are based on a return period substantially greater than the 50-year return period used in the past. Thus, traditional perceptions of safety may become somewhat more obscure or even confused with the more recent changes to the design process. It is also important to remember that the return period of the design load is not the only factor determining safety; the selection of safety factors (ASD) and load factors (LRFD) depends on the definition of a nominal design load (i.e., its return period) and the material's strength characterization to achieve a specified level of safety.

2.5.2 Basic Safety Concepts in Allowable Stress Design

The concept of ASD is demonstrated in a generic design equation or performance function (see Equation 2.5-1). In traditional allowable stress design, it is common to divide the characteristic (i.e., fifth percentile) material strength value by a safety factor of greater than 1 to determine an allowable design strength dependent on a selected limit state (i.e., proportional limit or rupture) and material type, among other factors that involve the judgment of specification-

[1]The apparent lack of agreement between a few long-term wind speed records beckons a more thorough validation of hurricane risk models and predicted design wind speeds along the Gulf and Atlantic coasts (Rosowsky and Cheng, 1999).

writing groups. The allowable design strength is then compared to the stresses created by a nominal design load combination, usually based on a 50-year mean recurrence interval event. A lower safety factor is generally applied to design conditions that are less variable or that are associated with a "noncritical" consequence, while the higher safety factor is typically applied to elements associated with greater uncertainty, such as connections. In addition, a higher safety factor is usually selected for materials, systems, or stress conditions that result in an abrupt failure mode without warning. Recognizing the impracticality of introducing a safety factor for each load type, the safety factor is also intended to cover the variability in loads.

Equation 2.5-1

$$\frac{R}{S.F.} \geq L$$

where,

R = nominal resistance (or design stress), usually based on the fifth percentile strength property of interest (also known as the characteristic strength value)
S.F. = the safety factor (R/S.F. is known as the allowable stress)
L = the load effect caused by the nominal design load combination (in units of R)

The equation refers to characteristic material strength, which represents the material stress value used for design purposes (also known as nominal or design strength or stress). When characteristic material strength (normalized to standard conditions) is divided by a safety factor, the result is an allowable material strength or stress. Given that materials exhibit variability in their stress capacity (some more variable than others), it is necessary to select a statistical value from the available material test data. Generally, though not always, the test methods, data, and evaluations of characteristic material strength values follow standardized procedures that vary across material industries (i.e., concrete, wood, steel, etc.) due in part to the uniqueness of each material. In most cases, the characteristic strength value is based on a lower-bound test statistic such as the fifth percentile, which is a value at which no more than 5 percent of the material specimens from a sample exhibit a lesser value. Since sampling is involved, the sampling methodology and sample size become critical to confidence in the characteristic strength value for general design applications.

In some cases, procedures for establishing characteristic material strength values are highly sophisticated and address many of the concerns mentioned above; in other cases, the process is simple and involves reduced levels of exactness or confidence (i.e., use of the lowest value in a small number of tests). Generally, the more variable a material, the more sophisticated the determination of characteristic material strength properties. A good example is the wood industry, whose many species and grades of lumber further complicate the inherent nonhomogenity of the product. Therefore, the wood industry uses fairly sophisticated procedures to sample and determine strength properties for a multitude of material conditions and properties (see Chapter 5).

Obviously, increasing the safety factor enhances the level of safety achieved in ASD (see Table 2.2 for the effect of varying safety factors to resist

wind loads in a typical hurricane-prone wind environment). The level of safety in Table 2.2 is presented as the probability of exceeding the characteristic material, connection, or assembly strength (i.e., fifth percentile strength value) over a 50-year reference period. While Table 2.2 is a nonconventional representation of safety, it demonstrates that an increase in the safety factor has a disproportionate effect on the level of safety achieved in terms of reducing the probability of failure. For example, increasing the safety factor substantially above 1 eventually begins to yield diminishing returns in terms of safety benefits. Clearly, the sensitivity of safety to adjustments in the safety factor is not a linear relationship (i.e., doubling the safety factor does not double safety). For this and other reasons, decisions regarding safety are embodied in the various material design specifications used by designers.

TABLE 2.2 Effect of Safety Factor on Level of Safety in ASD for a Typical Hurricane-Prone Wind Climate[1]

A	B	C	D	E	F
ASD Safety Factor	**Equivalent Wind Speed Factor ($\sqrt{A}$)**	**Design Wind Speed (mph gust)**	**'Ultimate' Event Wind Speed B x C (mph, gust)**	**'Ultimate' Event Return Period (years)**	**Chance of Exceedance in a 50-Year Period**
1.0	1.00	120	120	50	63.46%
2.0	1.41	120	170	671	7.18%
3.0	1.73	120	208	4,991	1.00%
4.0	2.00	120	240	27,318	0.18%

Note:
[1]The "ultimate" event is determined by multiplying the design (i.e., 50-year return period) wind speed by the square root of the safety factor. The derivation is based on multiplying both sides of Equation 2.5-1 by the safety factor and realizing that the wind load is related to the wind speed squared. Thus, the design or performance check is transformed to one with a safety factor of 1, but the load (or event) is increased to a higher return period to maintain an equivalent performance function.

As represented in current material design specifications and building code provisions, the ASD safety factors are the product of theory, past experience, and judgment and are intended for general design purposes. As such, they may not be specially "tuned" for specific applications such as housing. Further, various material specifications and standards vary in their treatment of safety factors and associated levels of safety (i.e., target safety).

2.5.3 Basic Safety Concepts in Load and Resistance Factor Design

The LRFD format has been conservatively calibrated to the level of safety represented by past ASD design practice and thus retains a tangible connection with historically accepted norms of structural safety (Galambos et al., 1982; Ellingwood et al., 1982; and others).[2] Thus, a similar level of safety is achieved with either method. However, the LRFD approach uses two factors–one applied

[2]It should be noted that historically accepted performance of wood-framed design, particularly housing, has not been specially considered in the development of modern LRFD design provisions for wood or other materials (i.e., concrete in foundations).

to the load and one applied to the resistance or strength property–that permits a more consistent treatment of safety across a broader range of design conditions.

Equation 2.5-2 shows conceptually the LRFD design format (i.e., performance function) and compares a factored characteristic resistance value with a factored nominal load. Thus, for a given hazard condition and given material, and similar to the outcome described in the previous section on ASD, increasing the load factor and/or decreasing the resistance factor has the effect of increasing the level of safety. Figure 2.5 depicts the variable nature of building loads and resistance and the safety margin relative to design loads and nominal resistance.

Equation 2.5-2

$$\phi R \geq \sum \gamma L$$

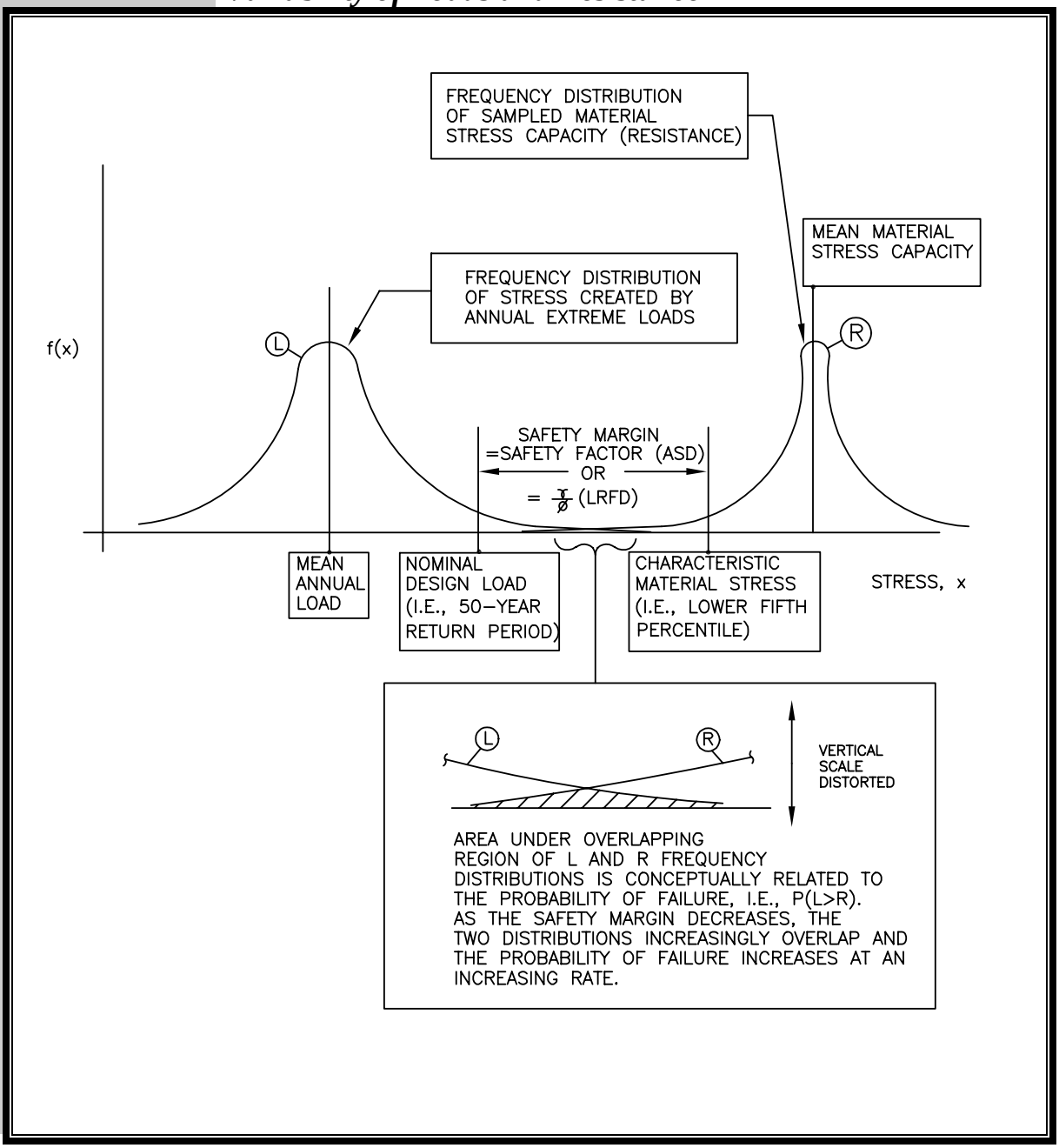
where,

ϕ = resistance factor (phi)
R = nominal resistance or design stress usually based on the fifth percentile strength property of interest (also known as the characteristic strength value)
γ = load factor for each load in a given load combination (gamma)
L = the stress created by each load in a nominal design load combination (in units of R)

A resistance factor is applied to a characteristic material strength value to account for variability in material strength properties. The resistance factor generally ranges from 0.5 to 0.9, with the lower values applicable to those strength properties that have greater variability or that are associated with an abrupt failure that gives little warning. The resistance factor also depends on the selected characterization of the nominal or characteristic strength value for design purposes (i.e., average, lower fifth percentile, lowest value of a limited number of tests, etc.).

A load factor is individually applied to each load in a nominal design load combination to account for the variability and nature of the hazard or combined hazards. It also depends on the selected characterization of the nominal load for design purposes (i.e., 50-year return period, 475-year return period, or others). In addition, the load factors proportion the loads relative to each other in a combination of loads (i.e., account for independence or correlation between loads and their likely "point-in-time" values when one load assumes a maximum value). Thus, the load factor for a primary load in a load combination may range from 1 to 1.6 in LRFD. For other transient loads in a combination, the factors are generally much less than 1. In this manner, the level of safety for a given material and nominal design load is determined by the net effect of factors–one on the resistance side of the design equation and the others on the load side. For ASD, the factors and their purpose are embodied in one simple factor–the safety factor.

FIGURE 2.5 ***Basic Concept of Safety in LRFD and ASD Considering the Variability of Loads and Resistance***



2.5.4 Putting Safety into Perspective

As discussed in Section 2.5, there is no absolute measure of safety. Therefore, the theory used to quantify safety is, at best, a relative measure that must be interpreted in consideration of the many assumptions underlying the treatment of uncertainty in the design process. Any reliable measure of safety must look to past experience and attempt to evaluate historic data in a rational manner to predict the future. Some indication of past experience with respect to housing performance was discussed in Chapter 1. However, it is important to

understand the risk associated with structural failures relative to other sources of risk. It is also instructive to understand the economic significance of damage to a structure as it, too, is a particular consequence of risk that may be associated with design decisions, even though it is beyond the primary concern of life-safety. Economic consequences are becoming increasingly debated and influential in the development of codified guidelines for structural design. Thus, some engineering requirements in codes may address two very different objectives–one being life-safety and the other being property protection or damage reduction. Finally, the manner in which these two different forms of risk are presented can have a profound impact on the perspective of risk and the perceived need for action or inaction.

Natural disasters and other events that affect buildings are given great attention in the media. In part, this attention is due to the relative infrequency of catastrophic (i.e., life-threatening) failures of buildings (such as homes) as compared to other consumer risks. Table 2.3 lists various risks and the associated estimates of mortality (i.e., life-safety). As illustrated in the data of Table 2.3, building related failures present relatively low risk in comparison to other forms of consumer risks. In fact, the risk associated with auto accidents is about two to three orders of magnitude greater than risks associated with building structural failures and related extreme loads. Also, the data must be carefully interpreted relative to a particular design objective and the ability to effectively address the risk through design solutions. For example, most deaths in hurricanes are related to flooding and indirect trauma following an event. These deaths are not related to wind damage to the structure. In fact, the number of deaths related to hurricane wind damage to houses is likely to be less than 10 persons in any given year and, of these, only a few may be eliminated by reasonable alterations of building design or construction practices. On the other hand, deaths due to flooding may be best resolved by improved land management practices and evacuation. A similar breakdown can be applied to other structural life-safety risks in Table 2.3.

TABLE 2.3 Commonplace Risks and Mortality Rates [1]

Commonplace Risks	Mean Annual Mortality Risk (average per capita)	Estimated Annual Mortality[2]
Smoking	3.6×10^{-3}	1,000,000
Cancer	2.8×10^{-3}	800,000
Auto accidents	2.4×10^{-4}	66,000
Homocide	1.0×10^{-4}	27,400
Fires	1.4×10^{-5}	3,800
Building collapse[3]	1.0×10^{-6}	N/A[3]
Lightening	5.0×10^{-7}	136
Tornadoes[4]	3.7×10^{-7}	100
Hurricanes[4]	1.5×10^{-7}	40
Earthquakes[5]	9.1×10^{-8}	25

Notes
[1]Data based on Wilson and Crouch, *Science*, 236 (1987) as reported by Ellingwood, *Structural Safety*, 13, Elsevier Science B.V. (1994) except as noted.
[2]Mortality rate based on October 1999 estimated population of 273,800,000 (U.S. Census)
[3]Annual probability is associated with building damage or failure, not the associated mortality.
[4]Data based on Golden and Snow, *Reviews of Geophysics*, 29, 4, November, 1991
[5]Data published in *Discover*, May 1996, p82 (original source unknown).

Property damage and insurance claims are also subject to significant media attention following building failures due to natural disasters and other extreme events. The conglomeration of economic impacts can indeed be staggering in appearance as shown in Table 2.4. However, the interpretation of the economic consequence must consider the appropriate application and perspective. For example, assuming that about 50 percent of insurance claims may be associated with housing damage and given that there are roughly 110,000,000 existing housing units in the United States, the total wind-related claims per housing unit in any given year may be about $32 (i.e., $7 million x 50 percent/110 million housing units). For a per unit national average, this loss is a small number. However, one must consider the disproportionate risk assumed by homes along the immediate hurricane coastlines which may experience more than an order of magnitude greater risk of damage (i.e., more than $320 per year of wind damage losses on average per housing unit). A similar break-down of economic loss can be made for other risks such as flooding and earthquakes.

TABLE 2.4 ***Annual Economic Losses of Insured Buildings Associated with Wind Damage*** [1]

Type of Wind Hazard	Annual Cost of Damage (all types of insured buildings)
Hurricanes	$5 billion[1]
Tornadoes	$1 billion[2]
Thunderstorm and other winds	$1 billion[3]

Notes:
[1]Data is based on Pielke and Landsea, *Weather and Forecasting*, September 1998 (data from 1925-1995, normalized to 1997 dollars). The normalized average has been relatively stable for the 70-year period of record. However, overall risk exposure has increased due to increasing population in hurricane-prone coastal areas.
[2]Data is based on National Research Council, *Facing the Challenge*, 1994.
[3]Data is based on a rough estimate from NCPI, 1993 for the period from 1986-1992.

While not a complete evaluation of life-safety data and economic loss data, the information in this section should establish a realistic basis for discerning the significance of safety and economic loss issues. Since engineers are often faced with the daunting task of balancing building initial cost with long term economic and life-safety consequences, a proper perspective on past experience is paramount to sound decision-making. In some cases, certain design decisions may affect insurance rates and other building ownership costs that should be considered by the designer.

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CHAPTER 3

Design Loads for Residential Buildings

3.1 General

Loads are a primary consideration in any building design because they define the nature and magnitude of hazards or external forces that a building must resist to provide reasonable performance (i.e., safety and serviceability) throughout the structure's useful life. The anticipated loads are influenced by a building's intended use (occupancy and function), configuration (size and shape), and location (climate and site conditions). Ultimately, the type and magnitude of design loads affect critical decisions such as material selection, construction details, and architectural configuration. Thus, to optimize the value (i.e., performance versus economy) of the finished product, it is essential to apply design loads realistically.

While the buildings considered in this guide are primarily single-family detached and attached dwellings, the principles and concepts related to building loads also apply to other similar types of construction, such as low-rise apartment buildings. In general, the design loads recommended in this guide are based on applicable provisions of the ASCE 7 standard–*Minimum Design Loads for Buildings and Other Structures* (ASCE, 1999). The ASCE 7 standard represents an acceptable practice for building loads in the United States and is recognized in virtually all U.S. building codes. For this reason, the reader is encouraged to become familiar with the provisions, commentary, and technical references contained in the ASCE 7 standard.

In general, the structural design of housing has not been treated as a unique engineering discipline or subjected to a special effort to develop better, more efficient design practices. Therefore, this part of the guide focuses on those aspects of ASCE 7 and other technical resources that are particularly relevant to the determination of design loads for residential structures. The guide provides supplemental design assistance to address aspects of residential construction where current practice is either silent or in need of improvement. The guide's

methods for determining design loads are complete yet tailored to typical residential conditions. As with any design function, the designer must ultimately understand and approve the loads for a given project as well as the overall design methodology, including all its inherent strengths and weaknesses. Since building codes tend to vary in their treatment of design loads the designer should, as a matter of due diligence, identify variances from both local accepted practice and the applicable building code relative to design loads as presented in this guide, even though the variances may be considered technically sound.

Complete design of a home typically requires the evaluation of several different types of materials as in Chapters 4 through 7. Some material specifications use the allowable stress design (ASD) approach while others use load and resistance factor design (LRFD). *Chapter 4 uses the LRFD method for concrete design and the ASD method for masonry design. For wood design, Chapters 5, 6, and 7 use ASD.* Therefore, for a single project, it may be necessary to determine loads in accordance with both design formats. This chapter provides load combinations intended for each method. The determination of individual nominal loads is essentially unaffected. Special loads such as flood loads, ice loads, and rain loads are not addressed herein. The reader is referred to the ASCE 7 standard and applicable building code provisions regarding special loads.

3.2 Load Combinations

The load combinations in Table 3.1 are recommended for use with design specifications based on allowable stress design (ASD) and load and resistance factor design (LRFD). Load combinations provide the basic set of building load conditions that should be considered by the designer. They establish the proportioning of multiple transient loads that may assume point-in-time values when the load of interest attains its extreme design value. Load combinations are intended as a guide to the designer, who should exercise judgment in any particular application. The load combinations in Table 3.1 are appropriate for use with the design loads determined in accordance with this chapter.

The principle used to proportion loads is a recognition that when one load attains its maximum life-time value, the other loads assume arbitrary point-in-time values associated with the structure's normal or sustained loading conditions. The advent of LRFD has drawn greater attention to this principle (Ellingwood et al., 1982; Galambos et al., 1982). The proportioning of loads in this chapter for allowable stress design (ASD) is consistent with and normalized to the proportioning of loads used in newer LRFD load combinations. However, this manner of proportioning ASD loads has seen only limited use in current code-recognized documents (AF&PA, 1996) and has yet to be explicitly recognized in design load specifications such as ASCE 7. ASD load combinations found in building codes have typically included some degree of proportioning (i.e., D + W + 1/2S) and have usually made allowance for a special reduction for multiple transient loads. Some earlier codes have also permitted allowable material stress increases for load combinations involving wind and earthquake loads. None of these adjustments for ASD load combinations is recommended for use with Table 3.1 since the load proportioning is considered sufficient.

It should also be noted that the wind load factor of 1.5 in Table 3.1 used for load and resistant factor design is consistent with traditional wind design practice (ASD and LRFD) and has proven adequate in hurricane-prone environments when buildings are properly designed and constructed. The 1.5 factor is equivalent to the earlier use of a 1.3 wind load factor in that the newer wind load provisions of ASCE 7-98 include separate consideration of wind directionality by adjusting wind loads by an explicit wind directionality factor, K_D, of 0.85. Since the wind load factor of 1.3 included this effect, it must be adjusted to 1.5 in compensation for adjusting the design wind load instead (i.e., 1.5/1.3 = 0.85). The 1.5 factor may be considered conservative relative to traditional design practice in nonhurricane-prone wind regions as indicated in the calibration of the LRFD load factors to historic ASD design practice (Ellingwood et al., 1982; Galambos et al., 1982). In addition, newer design wind speeds for hurricane-prone areas account for variation in the extreme (i.e., long return period) wind probability that occurs in hurricane hazard areas. Thus, the return period of the design wind speeds along the hurricane-prone coast varies from roughly a 70- to 100-year return period on the wind map in the 1998 edition of ASCE 7 (i.e., not a traditional 50-year return period wind speed used for the remainder of the United States). The latest wind design provisions of ASCE 7 include many advances in the state of the art, but the ASCE commentary does not clearly describe the condition mentioned above in support of an increased wind load factor of 1.6 (ASCE, 1999). Given that the new standard will likely be referenced in future building codes, the designer may eventually be required to use a higher wind load factor for LRFD than that shown in Table 3.1. The above discussion is intended to help the designer understand the recent departure from past successful design experience and remain cognizant of its potential future impact to building design.

The load combinations in Table 3.1 are simplified and tailored to specific application in residential construction and the design of typical components and systems in a home. These or similar load combinations are often used in practice as short-cuts to those load combinations that govern the design result. This guide makes effective use of the short-cuts and demonstrates them in the examples provided later in the chapter. The short-cuts are intended only for the design of residential light-frame construction.

TABLE 3.1 ***Typical Load Combinations Used for the Design of Components and Systems***[1]

Component or System	ASD Load Combinations	LRFD Load Combinations
Foundation wall (gravity and soil lateral loads)	D + H D + H + L^2 + 0.3(L_r + S) D + H + (L_r or S) + 0.3L^2	1.2D + 1.6H 1.2D + 1.6H + 1.6L^2 + 0.5(L_r + S) 1.2D + 1.6H + 1.6(L_r or S) + 0.5L^2
Headers, girders, joists, interior load-bearing walls and columns, footings (gravity loads)	D + L^2 + 0.3 (L_r or S) D + (L_r or S) + 0.3 L^2	1.2D + 1.6L^2 + 0.5 (L_r or S) 1.2D + 1.6(L_r or S) + 0.5 L^2
Exterior load-bearing walls and columns (gravity and transverse lateral load)[3]	Same as immediately above plus D + W D + 0.7E + 0.5L^2 + 0.2S[4]	Same as immediately above plus 1.2D + 1.5W 1.2D + 1.0E + 0.5L^2 + 0.2S[4]
Roof rafters, trusses, and beams; roof and wall sheathing (gravity and wind loads)	D + (L_r or S) 0.6D + W_u[5] D + W	1.2D + 1.6(L_r or S) 0.9D + 1.5W_u[5] 1.2D + 1.5W
Floor diaphragms and shear walls (in-plane lateral and overturning loads)[6]	0.6D + (W or 0.7E)	0.9D + (1.5W or 1.0E)

Notes:
[1]The load combinations and factors are intended to apply to nominal design loads defined as follows: D = estimated mean dead weight of the construction; H = design lateral pressure for soil condition/type; L = design floor live load; L_r = maximum roof live load anticipated from construction/maintenance; W = design wind load; S = design roof snow load; and E = design earthquake load. The design or nominal loads should be determined in accordance with this chapter.
[2]Attic loads may be included in the floor live load, but a 10 psf attic load is typically used only to size ceiling joists adequately for access purposes. However, if the attic is intended for storage, the attic live load (or some portion) should also be considered for the design of other elements in the load path.
[3]The transverse wind load for stud design is based on a localized component and cladding wind pressure; D + W provides an adequate and simple design check representative of worst-case combined axial and transverse loading. Axial forces from snow loads and roof live loads should usually not be considered simultaneously with an extreme wind load because they are mutually exclusive on residential sloped roofs. Further, in most areas of the United States, design winds are produced by either hurricanes or thunderstorms; therefore, these wind events and snow are mutually exclusive because they occur at different times of the year.
[4]For walls supporting heavy cladding loads (such as brick veneer), an analysis of earthquake lateral loads and combined axial loads should be considered. However, this load combination rarely governs the design of light-frame construction.
[5]W_u is wind uplift load from negative (i.e., suction) pressures on the roof. Wind uplift loads must be resisted by continuous load path connections to the foundation or until offset by 0.6D.
[6]The 0.6 reduction factor on D is intended to apply to the calculation of net overturning stresses and forces. For wind, the analysis of overturning should also consider roof uplift forces unless a separate load path is designed to transfer those forces.

3.3 Dead Loads

Dead loads consist of the permanent construction material loads comprising the roof, floor, wall, and foundation systems, including claddings, finishes, and fixed equipment. The values for dead loads in Table 3.2 are for commonly used materials and constructions in light-frame residential buildings. Table 3.3 provides values for common material densities and may be useful in calculating dead loads more accurately. The design examples in Section 3.10 demonstrate the straight-forward process of calculating dead loads.

TABLE 3.2 Dead Loads for Common Residential Construction[1]

Roof Construction			
Light-frame wood roof with wood structural panel sheathing and 1/2-inch gypsum board ceiling (2 psf) with asphalt shingle roofing (3 psf)		15 psf	
- with conventional clay/tile roofing		27 psf	
- with light-weight tile		21 psf	
- with metal roofing		14 psf	
- with wood shakes		15 psf	
- with tar and gravel		18 psf	
Floor Construction			
Light-frame 2x12 wood floor with 3/4-inch wood structural panel sheathing and 1/2-inch gypsum board ceiling (without 1/2-inch gypsum board, subtract 2 psf from all values) with carpet, vinyl, or similar floor covering		10 psf[2]	
- with wood flooring		12 psf	
- with ceramic tile		15 psf	
- with slate		19 psf	
Wall Construction			
Light-frame 2x4 wood wall with 1/2-inch wood structural panel sheathing and 1/2-inch gypsum board finish (for 2x6, add 1 psf to all values)		6 psf	
- with vinyl or aluminum siding		7 psf	
- with lap wood siding		8 psf	
- with 7/8-inch portland cement stucco siding		15 psf	
- with thin-coat-stucco on insulation board		9 psf	
- with 3-1/2-inch brick veneer		45 psf	
Interior partition walls (2x4 with 1/2-inch gypsum board applied to both sides)		6 psf	
Foundation Construction	Masonry[3]		Concrete
	Hollow	Solid or Full Grout	
6-inch-thick wall	28 psf	60 psf	75 psf
8-inch-thick wall	36 psf	80 psf	100 psf
10-inch-thick wall	44 psf	100 psf	123 psf
12-inch-thick wall	50 psf	125 psf	145 psf
6-inch x 12-inch concrete footing			73 plf
6-inch x 16-inch concrete footing			97 plf
8-inch x 24-inch concrete footing			193 plf

Notes:
[1]For unit conversions, see Appendix B.
[2]Value also used for roof rafter construction (i.e., cathedral ceiling).
[3]For partially grouted masonry, interpolate between hollow and solid grout in accordance with the fraction of masonry cores that are grouted.

TABLE 3.3 Densities for Common Residential Construction Materials[1]

Aluminum	170 pcf
Copper	556 pcf
Steel	492 pcf
Concrete (normal weight with light reinforcement)	145–150 pcf
Masonry, grout	140 pcf
Masonry, brick	100–130 pcf
Masonry, concrete	85–135 pcf
Glass	160 pcf
Wood (approximately 10 percent moisture content)[2]	
- spruce-pine-fir (G = 0.42)	29 pcf
- spruce-pine-fir, south (G = 0.36)	25 pcf
- southern yellow pine (G = 0.55)	38 pcf
- Douglas fir–larch (G = 0.5)	34 pcf
- hem-fir (G = 0.43)	30 pcf
- mixed oak (G = 0.68)	47 pcf
Water	62.4 pcf
Structural wood panels	
- plywood	36 pcf
- oriented strand board	36 pcf
Gypsum board	48 pcf
Stone	
- Granite	96 pcf
- Sandstone	82 pcf
Sand, dry	90 pcf
Gravel, dry	105 pcf

Notes:
[1]For unit conversions, see Appendix B.
[2]The equilibrium moisture content of lumber is usually not more than 10 percent in protected building construction. The specific gravity, G, is the decimal fraction of dry wood density relative to that of water. Therefore, at a 10 percent moisture content, the density of wood is $1.1(G)(62.4\ lbs/ft^3)$. The values given are representative of average densities and may easily vary by as much as 15 percent depending on lumber grade and other factors.

3.4 Live Loads

Live loads are produced by the use and occupancy of a building. Loads include those from human occupants, furnishings, nonfixed equipment, storage, and construction and maintenance activities. Table 3.4 provides recommended design live loads for residential buildings. Example 3.1 in Section 3.10 demonstrates use of those loads and the load combinations specified in Table 3.1, along with other factors discussed in this section. As required to adequately define the loading condition, loads are presented in terms of uniform area loads (psf), concentrated loads (lbs), and uniform line loads (plf). The uniform and concentrated live loads should not be applied simultaneously in a structural evaluation. Concentrated loads should be applied to a small area or surface

consistent with the application and should be located or directed to give the maximum load effect possible in end-use conditions. For example, the stair concentrated load of 300 pounds should be applied to the center of the stair tread between supports. The concentrated wheel load of a vehicle on a garage slab or floor should be applied to all areas or members subject to a wheel or jack load, typically using a loaded area of about 20 square inches.

TABLE 3.4 Live Loads for Residential Construction[1]

Application	Uniform Load	Concentrated Load
Roof[2]		
Slope ≥ 4:12	15 psf	250 lbs
Flat to 4:12 slope	20 psf	250 lbs
Attic[3]		
With limited storage	10 psf	250 lbs
With storage	20 psf	250 lbs
Floors		
Bedroom areas[3,4]	30 psf	300 lbs
Other areas	40 psf	300 lbs
Garages	50 psf	2,000 lbs (vans, light trucks) 1,500 lbs (passenger cars)
Decks	40 psf	300 lbs
Balconies	60 psf	300 lbs
Stairs	40 psf	300 lbs
Guards and handrails	20 plf	200 lbs
Grab bars	N/A	250 lbs

Notes:
[1]Live load values should be verified relative to the locally applicable building code.
[2]Roof live loads are intended to provide a minimum load for roof design in consideration of maintenance and construction activities. They should not be considered in combination with other transient loads (i.e., floor live load, wind load, etc.) when designing walls, floors, and foundations. A 15 psf roof live load is recommended for residential roof slopes greater than 4:12; refer to ASCE 7-98 for an alternate approach.
[3]Loft sleeping and attic storage loads should be considered only in areas with a clear height greater than about 3 feet. The concept of a "clear height" limitation on live loads is logical, but it may not be universally recognized.
[4]Some codes require 40 psf for all floor areas.

The floor live load on any given floor area may be reduced in accordance with Equation 3.4-1 (Harris, Corotis, and Bova, 1980). The equation applies to floor and support members, such as beams or columns, that experience floor loads from a total tributary floor area greater than 200 square feet. This equation is different from that in ASCE 7-98 since it is based on data that applies to residential floor loads rather than commercial buildings.

[Equation 3.4-1]

$$L = L_o\left[0.25 + \frac{10.6}{\sqrt{A_t}}\right] \geq 0.75$$

where,

L = the adjusted floor live load for tributary areas greater than 200 square feet
A_t = the tributary from a single-story area assigned to a floor support member (i.e., girder, column, or footing)
L_o = the unreduced live load associated with a floor area of 200 ft^2 from Table 3.4

It should also be noted that the nominal design floor live load in Table 3.4 includes both a sustained and transient load component. The sustained component is that load typically present at any given time and includes the load associated with normal human occupancy and furnishings. For residential buildings, the mean sustained live load is about 6 psf but typically varies from 4 to 8 psf (Chalk, Philip, and Corotis, 1978). The mean transient live load for dwellings is also about 6 psf but may be as high as 13 psf. Thus, a total design live load of 30 to 40 psf is fairly conservative.

3.5 Soil Lateral Loads

The lateral pressure exerted by earth backfill against a residential foundation wall (basement wall) can be calculated with reasonable accuracy on the basis of theory, but only for conditions that rarely occur in practice (University of Alberta, 1992; Peck, Hanson, and Thornburn, 1974). Theoretical analyses are usually based on homogeneous materials that demonstrate consistent compaction and behavioral properties. Such conditions are rarely experienced in the case of typical residential construction projects.

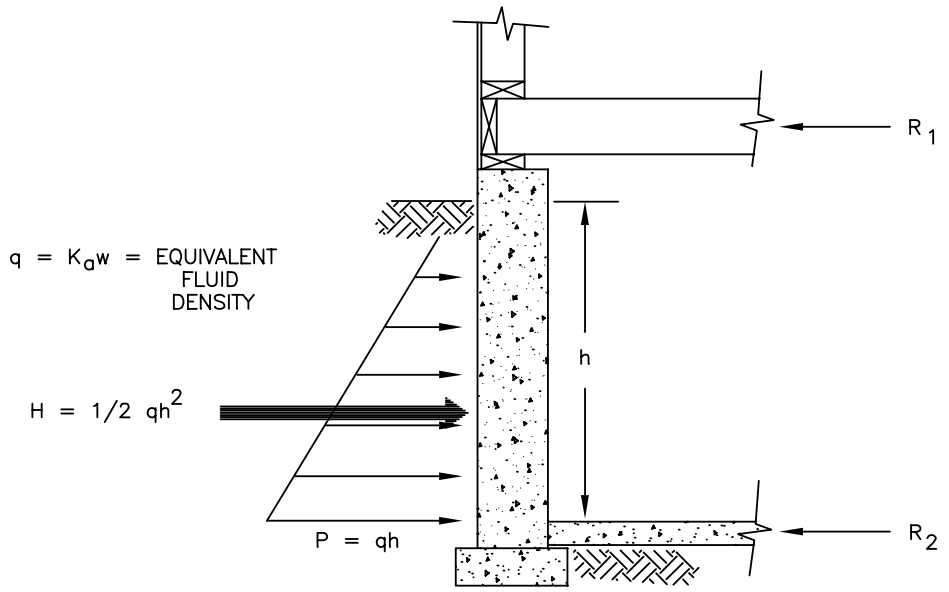
The most common method of determining lateral soil loads on residential foundations follows Rankine's (1857) theory of earth pressure and uses what is known as the Equivalent Fluid Density (EFD) method. As shown in Figure 3.1, pressure distribution is assumed to be triangular and to increase with depth.

In the EFD method, the soil unit weight w is multiplied by an empirical coefficient K_a to account for the fact that the soil is not actually fluid and that the pressure distribution is not necessarily triangular. The coefficient K_a is known as the active Rankine pressure coefficient. Thus, the equivalent fluid density (EFD) is determined as follows:

[Equation 3.5-1]

$$q = K_a w$$

FIGURE 3.1 ***Triangular Pressure Distribution on a Basement Foundation Wall***



It follows that for the triangular pressure distribution shown in Figure 3.1, the pressure at depth, h, in feet is

[Equation 3.5-2] $P = qh$

The total active soil force (pounds per lineal foot of wall length) is

[Equation 3.5-3] $$H = \frac{1}{2}(qh)(h) = \frac{1}{2}qh^2$$

where,

h = the depth of the unbalanced fill on a foundation wall
H = the resultant force (plf) applied at a height of h/3 from the base of the unbalanced fill since the pressure distribution is assumed to be triangular

The EFD method is subject to judgment as to the appropriate value of the coefficient K_a. The values of K_a in Table 3.5 are recommended for the determination of lateral pressures on residential foundations for various types of backfill materials placed with light compaction and good drainage. Given the long-time use of a 30 pcf equivalent fluid density in residential foundation wall prescriptive design tables (ICC, 1998), the values in Table 3.5 may be considered somewhat conservative for typical conditions. A relatively conservative safety factor of 3 to 4 is typically applied to the design of unreinforced or nominally reinforced masonry or concrete foundation walls (ACI 1999a and b). Therefore, at

imminent failure of a foundation wall, the 30 psf design EFD would correspond to an active soil lateral pressure determined by using an equivalent fluid density of about 90 to 120 pcf or more. The design examples in Chapter 4 demonstrate the calculation of soil loads.

TABLE 3.5 ***Values of K_a, Soil Unit Weight, and Equivalent Fluid Density by Soil Type***[1,2,3]

Type of Soil[4] (unified soil classification)	Active Pressure Coefficient (K_a)	Soil Unit Weight (pcf)	Equivalent Fluid Density (pcf)
Sand or gravel (GW, GP, GM, SW, SP)	0.26	115	30
Silty sand, silt, and sandy silt (GC, SM)	0.35	100	35
Clay-silt, silty clay (SM-SC, SC, ML, ML-CL)	0.45	100	45
Clay[5] (CL, MH, CH)	0.6	100	60

Notes:
[1]Values are applicable to well-drained foundations with less than 10 feet of backfill placed with light compaction or natural settlement as is common in residential construction. The values do not apply to foundation walls in flood-prone environments. In such cases, an equivalent fluid density value of 80 to 90 pcf would be more appropriate (HUD, 1977).
[2]Values are based on the *Standard Handbook for Civil Engineers*, Third Edition, 1983, and on research on soil pressures reported in *Thin Wall Foundation Testing*, Department of Civil Engineering, University of Alberta, Canada, March 1992. It should be noted that the values for soil equivalent fluid density differ from those recommended in ASCE 7-98 but are nonetheless compatible with current residential building codes, design practice, and the stated references.
[3]These values do not consider the significantly higher loads that can result from expansive clays and the lateral expansion of moist, frozen soil. Such conditions should be avoided by eliminating expansive clays adjacent to the foundation wall and providing for adequate surface and foundation drainage.
[4]Organic silts and clays and expansive clays are unsuitable for backfill material.
[5]Backfill in the form of clay soils (nonexpansive) should be used with caution on foundation walls with unbalanced fill heights greater than 3 to 4 feet and on cantilevered foundation walls with unbalanced fill heights greater than 2 to 3 feet.

Depending on the type and depth of backfill material and the manner of its placement, it is common practice in residential construction to allow the backfill soil to consolidate naturally by providing an additional 3 to 6 inches of fill material. The additional backfill ensures that surface water drainage away from the foundation remains adequate (i.e., the grade slopes away from the building). It also helps avoid heavy compaction that could cause undesirable loads on the foundation wall during and after construction. If soils are heavily compacted at the ground surface or compacted in lifts to standard Proctor densities greater than about 85 percent of optimum (ASTM, 1998), the standard 30 pcf EFD assumption may be inadequate. However, in cases where exterior slabs, patios, stairs, or other items are supported on the backfill, some amount of compaction is advisable unless the structures are supported on a separate foundation bearing on undisturbed ground.

3.6 Wind Loads

3.6.1 General

Wind produces nonstatic loads on a structure at highly variable magnitudes. The variation in pressures at different locations on a building is complex to the point that pressures may become too analytically intensive for precise consideration in design. Therefore, wind load specifications attempt to simplify the design problem by considering basic static pressure zones on a building representative of peak loads that are likely to be experienced. The peak pressures in one zone for a given wind direction may not, however, occur simultaneously with peak pressures in other zones. For some pressure zones, the peak pressure depends on a narrow range of wind direction. Therefore, the wind directionality effect must also be factored into determining risk-consistent wind loads on buildings. In fact, most modern wind load specifications take account of wind directionality and other effects in determining nominal design loads in some simplified form (SBCCI, 1999; ASCE, 1999). This section further simplifies wind load design specifications to provide an easy yet effective approach for designing typical residential buildings.

Because they vary substantially over the surface of a building, wind loads are considered at two different scales. On a large scale, the loads produced on the overall building, or on major structural systems that sustain wind loads from more than one surface of the building, are considered the main wind force-resisting system (MWFRS). The MWFRS of a home includes the shear walls and diaphragms that create the lateral force-resisting system (LFRS) as well as the structural systems such as trusses that experience loads from two surfaces (or pressure regimes) of the building. The wind loads applied to the MWFRS account for the large-area averaging effects of time-varying wind pressures on the surface or surfaces of the building.

On a smaller scale, pressures are somewhat greater on localized surface areas of the building, particularly near abrupt changes in building geometry (e.g., eaves, ridges, and corners). These higher wind pressures occur on smaller areas, particularly affecting the loads borne by components and cladding (e.g., sheathing, windows, doors, purlins, studs). The components and cladding (C&C) transfer localized time-varying loads to the MWFRS, at which point the loads average out both spatially and temporally since, at a given time, some components may be at near peak loads while others are at substantially less than peak.

The next section presents a simplified method for determining both MWFRS and C&C wind loads. Since the loads in Section 3.6.2 are determined for specific applications, the calculation of MWFRS and C&C wind loads is implicit in the values provided. Design Example 3.2 in Section 3.10 demonstrates the calculation of wind loads by applying the simplified method of the following Section 3.6.2 to several design conditions associated with wind loads and the load combinations presented in Table 3.1.

3.6.2 Determination of Wind Loads on Residential Buildings

The following method for the design of residential buildings is based on a simplification of the ASCE 7-98 wind provisions (ASCE, 1999); therefore, the wind loads are not an exact duplicate. Lateral loads and roof uplift loads are determined by using a projected area approach. Other wind loads are determined for specific components or assemblies that comprise the exterior building envelope. Five steps are required to determine design wind loads on a residential building and its components.

Step 1: Determine site design wind speed and basic velocity pressure

From the wind map in Figure 3.2 (refer to ASCE 7-98 for maps with greater detail), select a design wind speed for the site (ASCE, 1999). The wind speed map in ASCE 7-98 (Figure 3.2) includes the most accurate data and analysis available regarding design wind speeds in the United States. The new wind speeds may appear higher than those used in older design wind maps. The difference is due solely to the use of the "peak gust" to define wind speeds rather than an averaged wind speed as represented by the "fastest mile of wind" used in older wind maps. Nominal design peak gust wind speeds are typically 85 to 90 mph in most of the United States; however, along the hurricane-prone Gulf and Atlantic coasts, nominal design wind speeds range from 100 to 150 mph for the peak gust.

If relying on either an older fastest-mile wind speed map or older design provisions based on fastest-mile wind speeds, the designer should convert wind speed in accordance with Table 3.6 for use with this simplified method, which is based on peak gust wind speeds.

TABLE 3.6 Wind Speed Conversions

Fastest mile (mph)	70	75	80	90	100	110	120	130
Peak gust (mph)	85	90	100	110	120	130	140	150

Once the nominal design wind speed in terms of peak gust is determined, the designer can select the basic velocity pressure in accordance with Table 3.7. The basic velocity pressure is a reference wind pressure to which pressure coefficients are applied to determine surface pressures on a building. Velocity pressures in Table 3.7 are based on typical conditions for residential construction, namely, suburban terrain exposure and relatively flat or rolling terrain without topographic wind speed-up effects.

FIGURE 3.2 Basic Design Wind Speed Map from ASCE 7-98

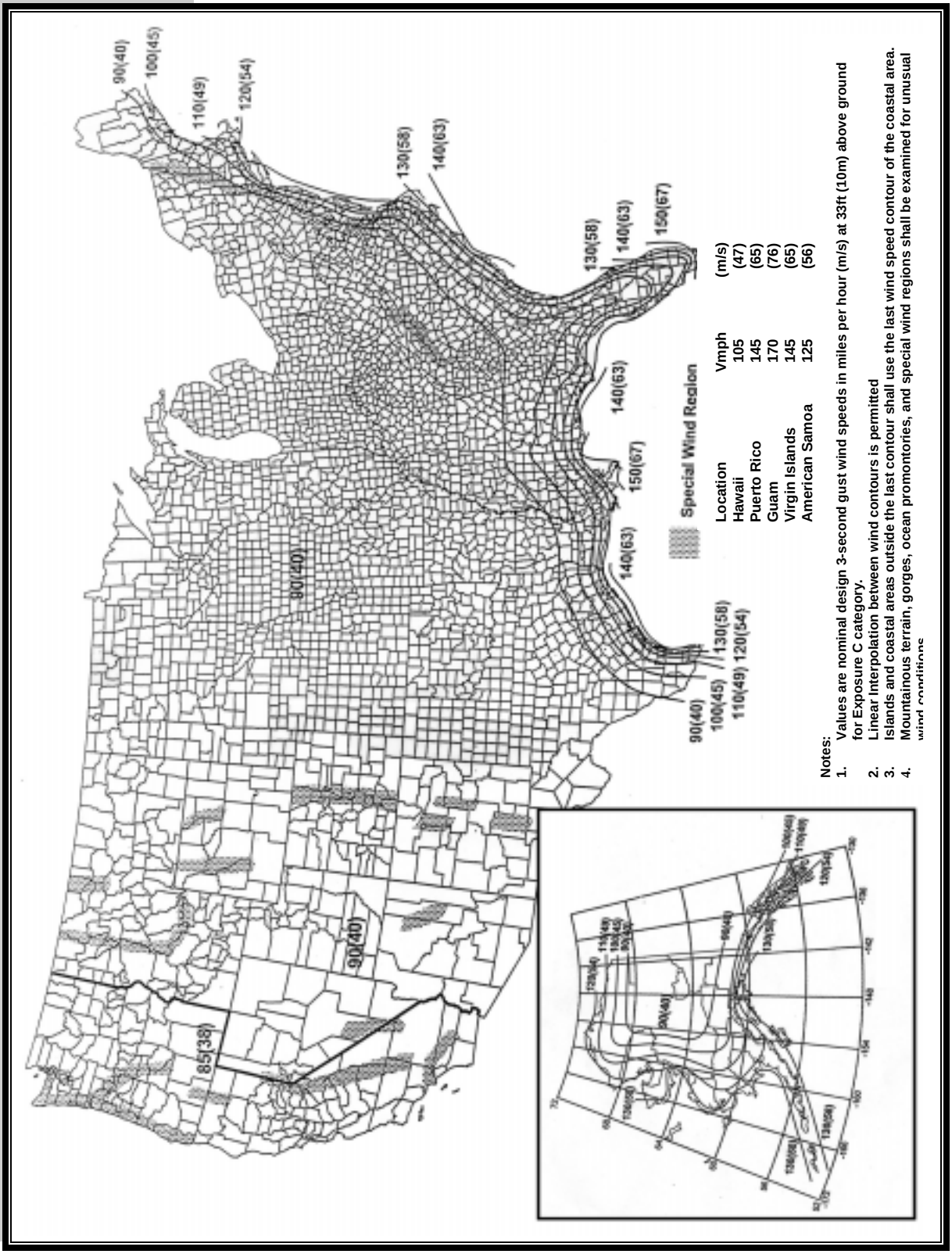


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TABLE 3.7 Basic Wind Velocity Pressures (psf) for Suburban Terrain[1]

Design Wind Speed, V (mph, peak gust)	One-Story Building ($K_Z = 0.6$)[2]	Two-Story Building ($K_Z = 0.67$)[2]	Three-Story Building ($K_Z = 0.75$)
85	9.4	10.5	11.8
90	10.6	11.8	13.2
100	13.1	14.6	16.3
110	15.8	17.6	19.7
120	18.8	21.0	23.5
130	22.1	24.6	27.6
140	25.6	28.6	32.0
150	29.4	32.8	36.7

Notes:
[1]Velocity pressure (psf) equals $0.00256\ K_D\ K_Z V^2$, where K_Z is the velocity pressure exposure coefficient associated with the vertical wind speed profile in suburban terrain at the mean roof height of the building. K_D is the wind directionality factor with a default value of 0.85.
[2]These two K_z factors are adjusted from that in ASCE 7 based on a recent study of the near-ground wind profile (NAHBRC, 1999). To be compliant with ASCE 7-98, a minimum K_z of 0.7 should be applied to determine velocity pressure for one-and two-story buildings in exposure B (suburban terrain) for the design of components and cladding only. For exposure C, the values are consistent with ASCE 7-98 and require no adjustment except that all tabulated values must be multiplied by 1.4 as described in Step 2.

Step 2: Adjustments to the basic velocity pressure

If appropriate, the basic velocity pressure from Step 1 should be adjusted in accordance with the factors below. The adjustments are cumulative.

Open exposure. The wind values in Table 3.7 are based on typical residential exposures to the wind. If a site is located in generally open, flat terrain with few obstructions to the wind in most directions or is exposed to a large body of water (i.e., ocean or lake), the designer should multiply the values in Table 3.7 by a factor of 1.4. The factor may be adjusted for sites that are considered intermediate to open suburban exposures. It may also be used to adjust wind loads according to the exposure related to the specific directions of wind approach to the building. The wind exposure conditions used in this guide are derived from ASCE 7-98 with some modification applicable to small residential buildings of three stories or less.

- Open terrain. Open areas with widely scattered obstructions, including shoreline exposures along coastal and noncoastal bodies of water.
- Suburban terrain. Suburban areas or other terrain with closely spaced obstructions that are the size of single-family dwellings or larger and extend in the upwind direction a distance no less than ten times the height of the building.

Protected exposure. If a site is generally surrounded by forest or densely wooded terrain with no open areas greater than a few hundred feet, smaller buildings such as homes experience significant wind load reductions from the typical suburban exposure condition assumed in Table 3.7. If such conditions exist and the site's design wind speed does not exceed about 120 mph peak gust, the designer may consider multiplying the values in Table 3.7 by 0.8. The factor may be used to adjust wind loads according to the exposure related to the specific directions of wind approach to the building. Wind load reductions associated with

a protected exposure in a suburban or otherwise open exposure have been shown to approximate 20 percent (Ho, 1992). In densely treed terrain with the height of the building below that of the tree tops, the reduction factor applied to Table 3.7 values can approach 0.6. The effect is known as shielding; however, it is not currently permitted by ASCE 7-98. Two considerations require judgment: Are the sources of shielding likely to exist for the expected life of the structure? Are the sources of shielding able to withstand wind speeds in excess of a design event?

Wind directionality. As noted, the direction of the wind in a given event does not create peak loads (which provide the basis for design pressure coefficients) simultaneously on all building surfaces. In some cases, the pressure zones with the highest design pressures are extremely sensitive to wind direction. In accordance with ASCE 7-98, the velocity pressures in Table 3.7 are based on a directionality adjustment of 0.85 that applies to hurricane wind conditions where winds in a given event are multidirectional but with varying magnitude. However, in "straight" wind climates, a directionality factor of 0.75 has been shown to be appropriate (Ho, 1992). Therefore, if a site is in a nonhurricane-prone wind area (i.e., design wind speed of 110 mph gust or less), the designer may also consider multiplying the values in Table 3.7 by 0.9 (i.e., 0.9 x 0.85 ≅ 0.75) to adjust for directionality effects in nonhurricane-prone wind environments. ASCE 7-98 currently does not recognize this additional adjustment to account for wind directionality in "straight" wind environments.

Topographic effects. If topographic wind speed-up effects are likely because a structure is located near the crest of a protruding hill or cliff, the designer should consider using the topographic factor provided in ASCE 7-98. Wind loads can be easily doubled for buildings sited in particularly vulnerable locations relative to topographic features that cause localized wind speed-up for specific wind directions (ASCE, 1999).

Step 3: Determine lateral wind pressure coefficients

Lateral pressure coefficients in Table 3.8 are composite pressure coefficients that combine the effect of positive pressures on the windward face of the building and negative (suction) pressures on the leeward faces of the building. When multiplied by the velocity pressure from Steps 1 and 2, the selected pressure coefficient provides a single wind pressure that is applied to the vertical projected area of the roof and wall as indicated in Table 3.8. The resulting load is then used to design the home's lateral force-resisting system (see Chapter 6). The lateral wind load must be determined for the two orthogonal directions on the building (i.e., parallel to the ridge and perpendicular to the ridge), using the vertical projected area of the building for each direction. Lateral loads are then assigned to various systems (e.g., shear walls, floor diaphragms, and roof diaphragms) by use of tributary areas or other methods described in Chapter 6.

TABLE 3.8 ***Lateral Pressure Coefficients for Application to Vertical Projected Areas***

Application	Lateral Pressure Coefficients
Roof Vertical Projected Area (by slope)	
Flat	0.0
3:12	0.3
6:12	0.5
≥9:12	0.8
Wall Projected Area	1.2

Step 4: Determine wind pressure coefficients for components and assemblies

The pressure coefficients in Table 3.9 are derived from ASCE 7-98 based on the assumption that the building is enclosed and not subject to higher internal pressures that may result from a windward opening in the building. The use of the values in Table 3.9 greatly simplifies the more detailed methodology described in ASCE 7-98; as a result, there is some "rounding" of numbers. With the exception of the roof uplift coefficient, all pressures calculated with the coefficients are intended to be applied to the perpendicular building surface area that is tributary to the element of concern. Thus, the wind load is applied perpendicular to the actual building surface, not to a projected area. The roof uplift pressure coefficient is used to determine a single wind pressure that may be applied to a horizontal projected area of the roof to determine roof tie-down connection forces.

For buildings in hurricane-prone regions subject to wind-borne debris, the GC_p values in Table 3.9 are required to be increased in magnitude by ±0.35 to account for higher potential internal pressures due to the possibility of a windward wall opening (i.e., broken window). The adjustment is not required by ASCE 7-98 in "wind-borne debris regions" if glazing is protected against likely sources of debris impact as shown by an "approved" test method; refer to Section 3.6.3.

Step 5: Determine design wind pressures

Once the basic velocity pressure is determined in Step 1 and adjusted in Step 2 for exposure and other site-specific considerations, the designer can calculate the design wind pressures by multiplying the adjusted basic velocity pressure by the pressure coefficients selected in Steps 3 and 4. The lateral pressures based on coefficients from Step 3 are applied to the tributary areas of the lateral force-resisting systems such as shear walls and diaphragms. The pressures based on coefficients from Step 4 are applied to tributary areas of members such as studs, rafters, trusses, and sheathing to determine stresses and connection forces.

TABLE 3-9 ***Wind Pressure Coefficients for Systems and Components (enclosed building)***[1]

Application	Pressure Coefficients (GC_p)[2]
Roof	
Trusses, roof beams, ridge and hip/valley rafters	-0.9, +0.4
Rafters and truss panel members	-1.2, +0.7
Roof sheathing	-2.2, +1.0
Skylights and glazing	-1.2, +1.0
Roof uplift[3]	
- hip roof with slope between 3:12 and 6:12	-0.9
- hip roof with slope greater than 6:12	-0.8
- all other roof types and slopes	-1.0
Windward overhang[4]	+0.8
Wall	
All framing members	-1.2, +1.1
Wall sheathing	-1.3, +1.2
Windows, doors, and glazing	-1.3, +1.2
Garage doors	-1.1, +1.0
Air-permeable claddings[5]	-0.9, 0.8

Notes:
[1]All coefficients include internal pressure in accordance with the assumption of an enclosed building. With the exception of the categories labeled trusses, roof beams, ridge and hip/valley rafters, and roof uplift, which are based on MWFRS loads, all coefficients are based on component with cladding wind loads.
[2]Positive and negative signs represent pressures acting inwardly and outwardly, respectively, from the building surface. A negative pressure is a suction or vacuum. Both pressure conditions should be considered to determine the controlling design criteria.
[3]The roof uplift pressure coefficient is used to determine uplift pressures that are applied to the horizontal projected area of the roof for the purpose of determining uplift tie-down forces. Additional uplift force on roof tie-downs due to roof overhangs should also be included. The uplift force must be transferred to the foundation or to a point where it is adequately resisted by the dead load of the building and the capacity of conventional framing connections.
[4]The windward overhang pressure coefficient is applied to the underside of a windward roof overhang and acts upwardly on the bottom surface of the roof overhang. If the bottom surface of the roof overhang is the roof sheathing or the soffit is not covered with a structural material on its underside, then the overhang pressure shall be considered additive to the roof sheathing pressure.
[5]Air-permeable claddings allow for pressure relief such that the cladding experiences about two-thirds of the pressure differential experienced across the wall assembly (FPL, 1999). Products that experience reduced pressure include lap-type sidings such as wood, vinyl, aluminum, and other similar sidings. Since these components are usually considered "nonessential," it may be practical to multiply the calculated wind load on any nonstructural cladding by 0.75 to adjust for a serviceability wind load (Galambos and Ellingwood, 1986). Such an adjustment would also be applicable to deflection checks, if required, for other components listed in the table. However, a serviceability load criterion is not included or clearly defined in existing design codes.

3.6.3 Special Considerations in Hurricane-Prone Environments

3.6.3.1 Wind-Borne Debris

The wind loads determined in the previous section assume an enclosed building. If glazing in windows and doors is not protected from wind-borne debris or otherwise designed to resist potential impacts during a major hurricane, a building is more susceptible to structural damage owing to higher internal building pressures that may develop with a windward opening. The potential for water damage to building contents also increases. Openings formed in the building envelope during a major hurricane or tornado are often related to unprotected glazing, improperly fastened sheathing, or weak garage doors and their attachment to the building. Section 3.9 briefly discusses tornado design conditions.

Recent years have focused much attention on wind-borne debris but with comparatively little scientific direction and poorly defined goals with respect to safety (i.e., acceptable risk), property protection, missile types, and reasonable impact criteria. Conventional practice in residential construction has called for simple plywood window coverings with attachments to resist the design wind loads. In some cases, homeowners elect to use impact-resistant glazing or shutters. Regardless of the chosen method and its cost, the responsibility for protection against wind-borne debris has traditionally rested with the homeowner. However, wind-borne debris protection has recently been mandated in some local building codes.

Just what defines impact resistance and the level of impact risk during a hurricane has been the subject of much debate. Surveys of damage following major hurricanes have identified several factors that affect the level of debris impact risk, including

- wind climate (design wind speed);
- exposure (e.g., suburban, wooded, height of surrounding buildings);
- development density (i.e., distance between buildings);
- construction characteristics (e.g., type of roofing, degree of wind resistance); and
- debris sources (e.g., roofing, fencing, gravel, etc.).

Current standards for selecting impact criteria for wind-borne debris protection do not explicitly consider all of the above factors. Further, the primary debris source in typical residential developments is asphalt roof shingles, which are not represented in existing impact test methods. These factors can have a dramatic effect on the level of wind-borne debris risk; moreover, existing impact test criteria appear to take a worst-case approach. Table 3.10 presents an example of missile types used for current impact tests. Additional factors to consider include emergency egress or access in the event of fire when impact-resistant glazing or fixed shutter systems are specified, potential injury or misapplication during installation of temporary methods of window protection, and durability of protective devices and connection details (including installation quality) such that they themselves do not become a debris hazard over time.

TABLE 3.10 Missile Types for Wind-Borne Debris Impact Tests[1,2]

Description	Velocity	Energy
2-gram steel balls	130 fps	10 ft-lb
4.5-lb 2x4	40 fps	100 ft-lb
9.0-lb 2x4	50 fps	350 ft-lb

Notes:
[1]Consult ASTM E 1886 (ASTM, 1997) or SSTD 12-97 (SBCCI, 1997) for guidance on testing apparatus and methodology.
[2]These missile types are not necessarily representative of the predominant types or sources of debris at any particular site. Steel balls are intended to represent small gravels that would be commonly used for roof ballast. The 2x4 missiles are intended to represent a direct, end-on blow from construction debris without consideration of the probability of such an impact over the life of a particular structure.

In view of the above discussion, ASCE 7-98 identifies "wind-borne debris regions" as areas within hurricane-prone regions that are located (1) within one mile of the coastal mean high water line where the basic wind speed is equal to or greater than 110 mph or in Hawaii or (2) where the basic wind speed is equal to or greater than 120 mph. As described in Section 3.6.2, ASCE 7-98 requires higher internal pressures to be considered for buildings in wind-borne debris regions unless glazed openings are protected by impact-resistant glazing or protective devices proven as such by an approved test method. Approved test methods include ASTM E1886 and SSTD 12-97 (ASTM, 1997; SBCCI, 1997).

The wind load method described in Section 3.6.2 may be considered acceptable without wind-borne debris protection, provided that the building envelope (i.e., windows, doors, sheathing, and especially garage doors) is carefully designed for the required pressures. Most homes that experience wind-borne debris damage do not appear to exhibit more catastrophic failures, such as a roof blow-off, unless the roof was severely underdesigned in the first place (i.e., inadequate tie-down) or subject to poor workmanship (i.e., missing fasteners at critical locations). Those cases are often the ones cited as evidence of internal pressure in anecdotal field studies. However, garage doors that fail due to wind pressure more frequently precipitate additional damage related to internal pressure. Therefore, in hurricane-prone regions, garage door reinforcement or pressure-rated garage doors should be specified and their attachment to structural framing carefully considered.

3.6.3.2 Building Durability

Roof overhangs increase uplift loads on roof tie-downs and the framing members that support the overhangs. They do, however, provide a reliable means of protection against moisture and the potential decay of wood building materials. The designer should therefore consider the trade-off between wind load and durability, particularly in the moist, humid climate zones associated with hurricanes.

For buildings that are exposed to salt spray or mist from nearby bodies of salt water, the designer should also consider a higher-than-standard level of corrosion resistance for exposed fasteners and hardware. Truss plates near roof vents have also shown accelerated rates of corrosion in severe coastal exposures. The building owner, in turn, should consider a building maintenance plan that includes regular inspections, maintenance, and repair.

3.6.3.3 Tips to Improve Performance

The following design and construction tips are simple options for reducing a building's vulnerability to hurricane damage:

- One-story buildings are much less vulnerable to wind damage than two- or three-story buildings.

- On average, hip roofs have demonstrated better performance than gable-end roofs.

- Moderate roof slopes (i.e., 4:12 to 6:12) tend to optimize the trade-off between lateral loads and roof uplift loads (i.e., more aerodynamically efficient).

- Roof sheathing installation should be inspected for the proper type and spacing of fasteners, particularly at connections to gable-end framing.

- The installation of metal strapping or other tie-down hardware should be inspected as required to ensure the transfer of uplift loads.

- If composition roof shingles are used, high-wind fastening requirements should be followed (i.e., 6 nails per shingle in lieu of the standard 4 nails). A similar concern exists for tile roofing, metal roofing, and other roofing materials.

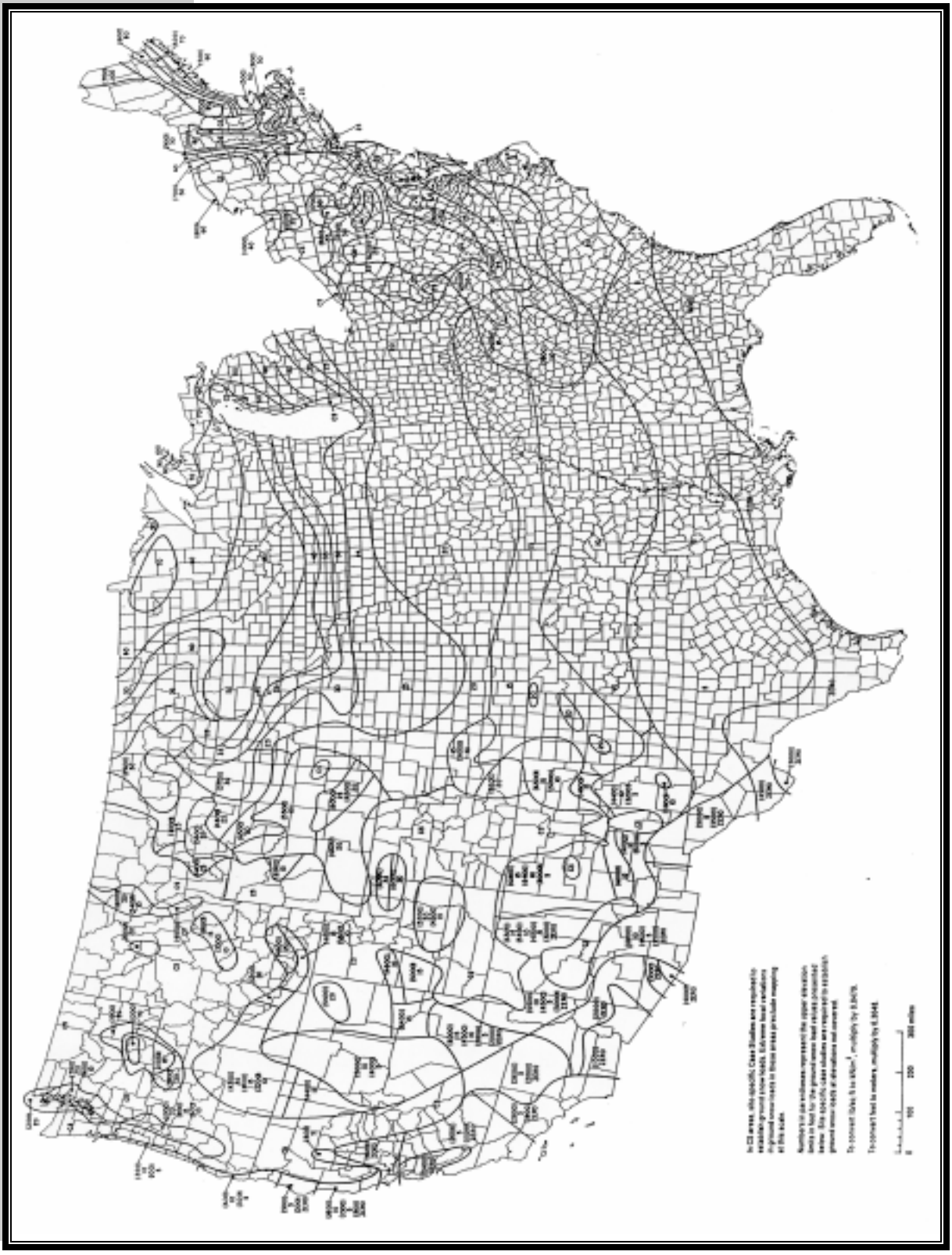
- Consider some practical means of glazed opening protection in the most severe hurricane-prone areas.

3.7 Snow Loads

For design purposes, snow is typically treated as a simple uniform gravity load on the horizontal projected area of a roof. The uniformly distributed design snow load on residential roofs can be easily determined by using the unadjusted ground snow load. This simple approach also represents standard practice in some regions of the United States; however, it does not account for a reduction in roof snow load that may be associated with steep roof slopes with slippery surfaces (refer to ASCE 7-98). To consider drift loads on sloped gable or hip roofs, the design roof snow load on the windward and leeward roof surfaces may be determined by multiplying the ground snow load by 0.8 and 1.2 respectively. In this case, the drifted side of the roof has 50 percent greater snow load than the non-drifted side of the roof. However, the average roof snow load is still equivalent to the ground snow load.

Design ground snow loads may be obtained from the map in Figure 3.3; however, snow loads are usually defined by the local building department. Typical ground snow loads range from 0 psf in the South to 50 psf in the northern United States. In mountainous areas, the ground snow load can surpass 100 psf such that local snow data should be carefully considered. In areas where the ground snow load is less than 15 psf, the minimum roof live load (refer to Section 3.4) is usually the controlling gravity load in roof design. For a larger map with greater detail, refer to ASCE 7-98.

FIGURE 3.3 Ground Snow Loads (ASCE 7-98)



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3.8 Earthquake Loads

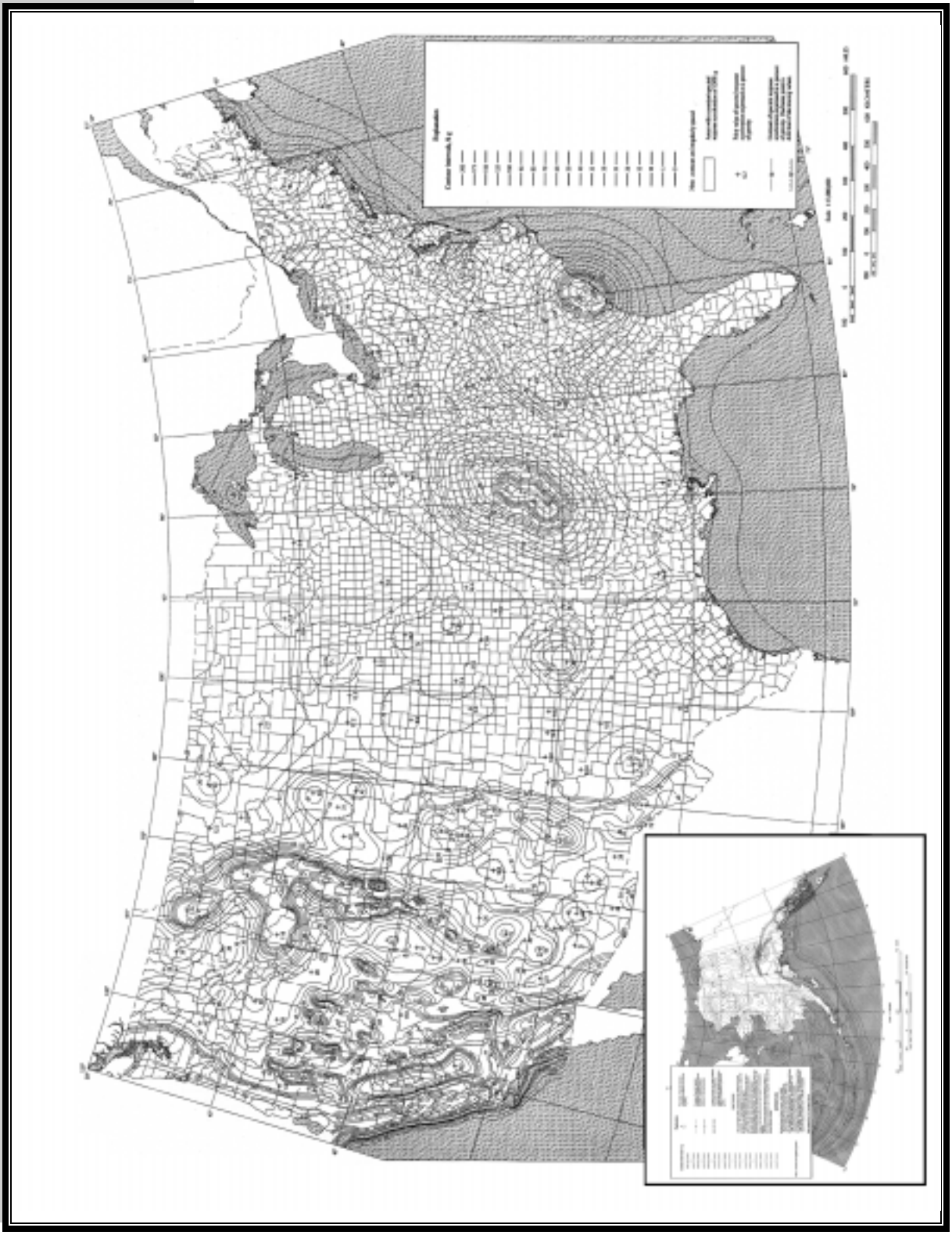
3.8.1 General

This section provides a simplified earthquake load analysis procedure appropriate for use in residential light-frame construction of not more than three stories above grade. As described in Chapter 2, the lateral forces associated with seismic ground motion are based on fundamental Newtonian mechanics (F = ma) expressed in terms of an equivalent static load. The method provided in this section is a simplification of the most current seismic design provisions (NEHRP, 1997[a and b]). It is also similar to a simplified approach found in more recent building code development (ICC, 1999).

Most residential designers use a simplified approach similar to that in older seismic design codes. The approach outlined in the next section follows the older approach in terms of its simplicity while using the newer seismic risk maps and design format of NEHRP-97 as incorporated into recent building code development efforts (ICC, 1999); refer to Figure 3.4. It should be noted, however, that the newer maps are not without controversy relative to seismic risk predictions, particularly in the eastern United States. For example, the newer maps are believed to overstate significantly the risk of earthquakes in the New Madrid seismic region around St. Louis, MO (Newman et al., 1999). Based on recent research and the manner of deriving the NEHRP-97 maps for the New Madrid seismic region, the design seismic loads may be conservative by a factor of 2 or more. The designer should bear in mind these uncertainties in the design process.

Chapter 1 discussed the performance of conventional residential construction in the Northridge Earthquake. In general, wood-framed homes have performed well in major seismic events, probably because of, among many factors, their light-weight and resilient construction, the strength provided by nonstructural systems such as interior walls, and their load distribution capabilities. Only in the case of gross absence of good judgment or misapplication of design for earthquake forces have severe life-safety consequences become an issue in light-frame, low-rise structures experiencing extreme seismic events.

FIGURE 3.4 ***Seismic Map of Design Short-Period Spectral Response Acceleration (g) (2 percent chance of exceedance in 50 years or 2,475-year return period)***



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3.8.2 Determination of Earthquake Loads on Houses

The total lateral force at the base of a building is called seismic base shear. The lateral force experienced at a particular story level is called the story shear. The story shear is greatest in the ground story and least in the top story. Seismic base shear and story shear (V) are determined in accordance with the following equation:

[Equation 3.8-1]

$$V = \frac{1.2\ S_{DS}}{R} W ,$$

where,

S_{DS} = the design spectral response acceleration in the short-period range determined by Equation 3.8-2 (g)
R = the response modification factor (dimensionless)
W = the total weight of the building or supported by the story under consideration (lb); 20 percent of the roof snow load is also included where the ground snow load exceeds 30 psf
1.2 = factor to increase the seismic shear load based on the belief that the simplified method may result in greater uncertainty in the estimated seismic load

When determining story shear for a given story, the designer attributes to that story one-half of the dead load of the walls on the story under consideration and the dead load supported by the story. Dead loads used in determining seismic story shear or base shear are found in Section 3.3. For housing, the interior partition wall dead load is reasonably accounted for by the use of a 6 psf load distributed uniformly over the floor area. When applicable, the snow load may be determined in accordance with Section 3.7. The inclusion of any snow load, however, is based on the assumption that the snow is always frozen solid and adhered to the building such that it is part of the building mass during the entire seismic event.

The design spectral response acceleration for short-period ground motion S_{DS} is typically used because light-frame buildings such as houses are believed to have a short period of vibration in response to seismic ground motion (i.e., high natural frequency). In fact, nondestructive tests of existing houses have confirmed the short period of vibration, although once ductile damage has begun to occur in a severe event, the natural period of the building likely increases. Chapter 1 discussed the apparent correlation between housing performance (degree of damage) and long-period (one-second) ground motion characteristics in the Northridge Earthquake (HUD, 1999). As yet, no valid methods are available to determine the natural period of vibration for use in the seismic design of light-frame houses. Therefore, the short-period ground motion is used in the interest of following traditional practice.

Values of S_s are obtained from Figure 3.7. For a larger map with greater detail, refer to ASCE 7-98. The value of S_{DS} should be determined in consideration of the mapped short-period spectral response acceleration S_s and the required soil site amplification factor F_a as follows:

[Equation 3.8-2]

$$S_{DS} = 2/3(S_s)(F_a)$$

The value of S_s ranges from practically zero in low-risk areas to 3g in the highest-risk regions of the United States. A typical value in high seismic areas is 1.5g. In general, wind loads control the design of the lateral force-resisting system of light-frame houses when S_s is less than about 1g. The 2/3 coefficient in Equation 3.8-2 is used to adjust to a design seismic ground motion value from that represented by the mapped S_s values (i.e., the mapped values are based on a "maximum considered earthquake" generally representative of a 2,475-year return period, with the design basis intended to represent a 475-year return period event).

Table 3.11 provides the values of F_a associated with a standard "firm" soil condition used for the design of residential buildings. F_a decreases with increasing ground motion because the soil begins to dampen the ground motion as shaking intensifies. Therefore, the soil can have a moderating effect on the seismic shear loads experienced by buildings in high seismic risk regions. Dampening also occurs between a building foundation and the soil and thus has a moderating effect. However, the soil-structure interaction effects on residential buildings have been the topic of little study; therefore, precise design procedures have yet to be developed. If a site is located on fill soils or "soft" ground, a different value of F_a should be considered. Nonetheless, as noted in the Anchorage Earthquake of 1964 and again 30 years later in the Northridge Earthquake (see Chapter 1), soft soils do not necessarily affect the performance of the above-ground house structure as much as they affect the site and foundations (e.g., settlement, fissuring, liquefaction, etc.).

TABLE 3.11 Site Soil Amplification Factor Relative to Acceleration (short period, firm soil)

S_s	≤ 0.25g	0.5g	0.75g	1.0g	≥ 1.25g
F_a	1.6	1.4	1.2	1.1	1.0

The seismic response modifier R has a long history in seismic design, but with little in the way of scientific underpinnings. In fact, it can be traced back to expert opinion in the development of seismic design codes during the 1950s (ATC, 1995). In recognition that buildings can effectively dissipate energy from seismic ground motions through ductile damage, the R factor was conceived to adjust the shear forces from that which would be experienced if a building could exhibit perfectly elastic behavior without some form of ductile energy dissipation. The concept has served a major role in standardizing the seismic design of buildings even though it has evolved in the absence of a repeatable and generalized evaluation methodology with a known relationship to actual building performance.

Those structural building systems that are able to withstand greater ductile damage and deformation without substantial loss of strength are assigned a higher value for R. The R factor also incorporates differences in dampening that are believed to occur for various structural systems. Table 3.12 provides some values for R that are relevant to residential construction.

TABLE 3.12 ***Seismic Response Modifiers for Residential Construction***

Structural System	Seismic Response Modifier, R[1]
Light-frame shear walls with wood structural panels used as bearing walls	6.0[2]
Light-frame shear walls with wall board/lath and plaster	2.0
Reinforced concrete shear walls[3]	4.5
Reinforced masonry shear walls[3]	3.5
Plain concrete shear walls	1.5
Plain masonry shear walls	1.25

Notes:
[1]The R-factors may vary for a given structural system type depending on wall configuration, material selection, and connection detailing, but these considerations are necessarily matters of designer judgment.
[2]The R for light-frame shear walls (steel-framed and wood-framed) with shear panels has been recently revised to 6 but is not yet published (ICC, 1999). Current practice typically uses an R of 5.5 to 6.5 depending on the edition of the local building code.
[3]The wall is reinforced in accordance with concrete design requirements in ACI-318 or ACI-530. Nominally reinforced concrete or masonry that has conventional amounts of vertical reinforcement such as one #5 rebar at openings and at 4 feet on center may use the value for reinforced walls provided the construction is no more than two stories above grade.

Design Example 3.3 in Section 3.10 demonstrates the calculation of design seismic shear load based on the simplified procedures. The reader is referred to Chapter 6 for additional information on seismic loads and analysis.

3.8.3 Seismic Shear Force Distribution

As described in the previous section, the *vertical distribution* of seismic forces to separate stories on a light-frame building is assumed to be in accordance with the mass supported by each story. However, design codes vary in the requirements related to vertical distribution of seismic shear. Unfortunately, there is apparently no clear body of evidence to confirm any particular method of vertical seismic force distribution for light-frame buildings. Therefore, in keeping with the simplified method given in Section 3.8.2, the approach used in this guide reflects what is considered conventional practice. The *horizontal distribution* of seismic forces to various shear walls on a given story also varies in current practice for light-frame buildings. In Chapter 6, several existing approaches to the design of the lateral force-resisting system of light-frame houses address the issue of horizontal force distribution with varying degrees of sophistication. Until methods of vertical and horizontal seismic force distribution are better understood for application to light-frame buildings, the importance of designer judgment cannot be overemphasized.

3.8.4 Special Seismic Design Considerations

Perhaps the single most important principle in seismic design is to ensure that the structural components and systems are adequately tied together to perform as a structural unit. Underlying this principle are a host of analytic challenges and uncertainties in actually defining what "adequately tied together" means in a repeatable, accurate, and theoretically sound manner.

Recent seismic building code developments have introduced several new factors and provisions that attempt to address various problems or uncertainties in the design process. Unfortunately, these factors appear to introduce as many

uncertainties as they address. Codes have tended to become more complicated to apply or decipher, perhaps detracting from some important basic principles in seismic design that, when understood, would provide guidance in the application of designer judgment. Many of the problems stem from the use of the seismic response modifier R which is a concept first introduced to seismic design codes in the 1950s (see discussion in previous section). Some of the issues and concerns are briefly described below based on a recent critique of seismic design approaches and other sources (ATC, 1995; NEHRP 1997a and b; ICBO, 1997).

Also known as "reserve strength," the concept of *overstrength* is a realization that a shear resisting system's ultimate capacity is usually significantly higher than required by a design load as a result of intended safety margins. At the same time, the seismic ground motion (load) is reduced by the R factor to account for ductile response of the building system, among other things. Thus, the actual forces experienced on various components (i.e. connections) during a design level event can be substantially higher, even though the resisting system may be able to effectively dissipate that force. Therefore, overstrength factors have been included in newer seismic codes with recommendations to assist in designing components that may experience higher forces than determined otherwise for the building lateral force resisting system using methods similar to Equation 3.8-1. It should be noted that current overstrength factors should not be considered exact and that actual values of overstrength can vary substantially.

In essence, the overstrength concept is an attempt to address the principle of balanced design. It strives to ensure that critical components, such as connections, have sufficient capacity so that the overall lateral force-resisting system is able to act in its intended ductile manner (i.e., absorbing higher-than-design forces). Thus, a premature failure of a critical component (i.e., a restraining connection failure) is avoided. An exact approach requires near-perfect knowledge about various connections, details, safety margins, and system-component response characteristics that are generally not available. However, the concept is extremely important and, for the most part, experienced designers have exercised this principle through a blend of judgment and rational analysis.

The concept of overstrength is addressed in Chapter 6 relative to the design of restraining connections for light-frame buildings by providing the designer with ultimate capacity values for light-frame shear wall systems. Thus, the designer is able to compare the unfactored shear wall capacity to that of hold-down restraints and other connections to ensure that the ultimate connection capacity is at least as much as that of the shear wall system. Some consideration of the ductility of the connection or component may also imply a response modification factor for a particular connection or framing detail. In summary, overstrength is an area where exact guidance does not exist and the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

The *redundancy factor* was postulated to address the reliability of lateral force-resisting systems by encouraging multiple lines of shear resistance in a building (ATC, 1995). It is now included in some of the latest seismic design provisions (NEHRP, 1997). Since it appears that redundancy factors have little technical basis and insufficient verification relative to light-frame structures (ATC, 1995), they are not explicitly addressed in this guide. In fact, residential buildings are generally recognized for their inherent redundancies that are

systematically overlooked when designating and defining a lateral force resisting system for the purpose of executing a rational design. However, the principle is important to consider. For example, it would not be wise to rely on one or two shear-resisting components to support a building. In typical applications of light-frame construction, even a single shear wall line has several individual segments and numerous connections that resist shear forces. At a minimum, there are two such shear wall lines in either orientation of the building, not to mention interior walls and other nonstructural elements that contribute to the redundancy of typical light-frame homes. In summary, redundancy is an area where exact guidance does not exist and the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

Deflection amplification has been applied in past and current seismic design codes to adjust the deflection or story drift determined by use of the design seismic shear load (as adjusted downward by the R factor) relative to that actually experienced without allowance for modified response (i.e., load not adjusted down by the R factor). For wood-framed shear wall construction, the deflection calculated at the nominal seismic shear load (Equation 3.8-1) is multiplied by a factor of 4 (NEHRP, 1997). Thus, the estimate of deflection or drift of the shear wall (or entire story) based on the design seismic shear load would be increased four-fold. Again, the conditions that lead to this level of deflection amplification and the factors that may affect it in a particular design are not exact (and are not obvious to the designer). As a result, conservative drift amplification values are usually selected for code purposes. Regardless, deflection or drift calculations are rarely applied in a residential (low-rise) wood-framed building design for three reasons. First, a methodology is not generally available to predict the drift behavior of light-frame buildings reliably and accurately. Second, the current design values used for shear wall design are relatively conservative and are usually assumed to provide adequate stiffness (i.e., limit drift). Third, code-required drift limits have not been developed for specific application to light-frame residential construction. Measures to estimate drift, however, are discussed in Chapter 6 in terms of nonlinear approximations of wood-frame shear wall load-drift behavior (up to ultimate capacity). In summary, deformation amplification is an area where exact guidance does not exist and predictive tools are unreliable. Therefore, the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

Another issue that has received greater attention in seismic design provisions is *irregularities*. Irregularities are related to special geometric or structural conditions that affect the seismic performance of a building and either require special design attention or should be altogether avoided. In essence, the presence of limits on structural irregularity speaks indirectly of the inability to predict the performance of a structure in a reliable, self-limiting fashion on the basis of analysis alone. Therefore, many of the irregularity limitations are based on judgment from problems experienced in past seismic events.

Irregularities are generally separated into plan and vertical structural irregularities. Plan structural irregularities include torsional imbalances that result in excessive rotation of the building, re-entrant corners creating “wings” of a building, floor or roof diaphragms with large openings or nonuniform stiffness, out-of-plane offsets in the lateral force resistance path, and nonparallel resisting systems. Vertical structural irregularities include stiffness irregularities (i.e., a

"soft" story), capacity irregularities (i.e., a "weak" story), weight (mass) irregularity (i.e., a "heavy" story), and geometric discontinuities affecting the interaction of lateral resisting systems on adjacent stories.

The concept of irregularities is associated with ensuring an adequate load path and limiting undesirable (i.e., hard to control or predict) building responses in a seismic event. Again, experienced designers generally understand the effect of irregularities and effectively address or avoid them on a case-by-case basis. For typical single-family housing, all but the most serious irregularities (i.e., "soft story") are generally of limited consequence, particularly given the apparently significant system behavior of light-frame homes (provided the structure is reasonably "tied together as a structural unit"). For larger structures, such as low- and high-rise commercial and residential construction, the issue of irregularity–and loads–becomes more significant. Because structural irregularities raise serious concerns and have been associated with building failures or performance problems in past seismic events, the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

A key issue related to building damage involves *deformation compatibility* of materials and detailing in a constructed system. This issue may be handled through specification of materials that have similar deformation capabilities or by system detailing that improves compatibility. For example, a relatively flexible hold-down device installed near a rigid sill anchor causes greater stress concentration on the more rigid element as evidenced by the splitting of wood sill plates in the Northridge Earthquake. The solution can involve increasing the rigidity of the hold-down device (which can lessen the ductility of the system, increase stiffness, and effectively increase seismic load) or redesigning the sill plate connection to accommodate the hold-down deformation and improve load distribution. As a nonstructural example of deformation compatibility, gypsum board interior finishes crack in a major seismic event well before the structural capability of the wall's structural sheathing is exhausted. Conversely, wood exterior siding and similar resilient finishes tend to deform compatibly with the wall and limit observable or unacceptable visual damage (HUD, 1994). A gypsum board interior finish may be made more resilient and compatible with structural deformations by using resilient metal channels or similar detailing; however, this enhancement has not yet been proven. Unfortunately, there is little definitive design guidance on deformation compatibility considerations in seismic design of wood-framed buildings and other structures.

As a final issue, it should be understood that the general objective of current and past seismic building code provisions has been to prevent collapse in extreme seismic events such that "protection of life is reasonably provided, but not with complete assurance" as stated in the 1990 *Blue Book* (SEAOC, 1990). It is often believed that damage can be controlled by use of a smaller R factor or, for a similar effect, a larger safety factor. Others have suggested using a higher design event. While either approach may indirectly reduce damage or improve performance, it does not necessarily improve the predictability of building performance and, therefore, may have uncertain benefits, if any, in many cases. However, some practical considerations as discussed above may lead to better-performing buildings, at least from the perspective of controlling damage.

3.9 Other Load Conditions

In addition to the loads covered in Sections 3.3 through 3.8 that are typically considered in the design of a home, other "forces of nature" may create loads on buildings. Some examples include

- frost heave;
- expansive soils;
- temperature effects; and
- tornadoes.

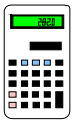
In certain cases, forces from these phenomena can drastically exceed reasonable design loads for homes. For example, frost heave forces can easily exceed 10,000 pounds per square foot (Linell and Lobacz, 1980). Similarly, the force of expanding clay soil can be impressive. In addition, the self-straining stresses induced by temperature-related expansion or contraction of a member or system that is restrained against movement can be very large, although they are not typically a concern in wood-framed housing. Finally, the probability of a direct tornado strike on a given building is much lower than considered practical for engineering and general safety purposes. The unique wind loads produced by an extreme tornado (i.e., F5 on the Fujita scale) may exceed typical design wind loads by almost an order of magnitude in effect. Conversely, most tornadoes have comparatively low wind speeds that can be resisted by attainable design improvements. However, the risk of such an event is still significantly lower than required by minimum accepted safety requirements.

It is common practice to avoid the above loads by using sound design detailing. For example, frost heave can be avoided by placing footings below a "safe" frost depth, building on nonfrost-susceptible materials, or using other frost protection methods (see Chapter 4). Expansive soil loads can be avoided by isolating building foundations from expansive soil, supporting foundations on a system of deep pilings, and designing foundations that provide for differential ground movements. Temperature effects can be eliminated by providing construction joints that allow for expansion and contraction. While such temperature effects on wood materials are practically negligible, some finishes such as ceramic tile can experience cracking when inadvertently restrained against small movements resulting from variations in temperature. Unfortunately, tornadoes cannot be avoided; therefore, it is not uncommon to consider the additional cost and protection of a tornado shelter in tornado-prone areas. A tornado shelter guide is available from the Federal Emergency Management Agency, Washington, DC.

As noted at the beginning of the chapter, this guide does not address loads from flooding, ice, rain, and other exceptional sources. The reader is referred to ASCE 7 and other resources for information regarding special load conditions (ASCE, 1999).

3.10 Design Examples

EXAMPLE 3.1 *Design Gravity Load Calculations and Use of ASD Load Combinations*



Given

- Three-story conventional wood-framed home
- 28' x 44' plan, clear-span roof, floors supported at mid-span
- Roof dead load = 15 psf (Table 3.2)
- Wall dead load = 8 psf (Table 3.2)
- Floor dead load = 10 psf (Table 3.2)
- Roof snow load = 16 psf (Section 3.7)
- Attic live load = 10 psf (Table 3.4)
- Second- and third-floor live load = 30 psf (Table 3.4)
- First-floor live load = 40 psf (Table 3.4)

Find

1. Gravity load on first-story exterior bearing wall
2. Gravity load on a column supporting loads from two floors

Solution

1. Gravity load on first-story exterior bearing wall

- Determine loads on wall

Dead load	=	roof DL + 2 wall DL + 2 floor DL
	=	1/2 (28 ft)(15 psf) + 2(8 ft)(8 psf) + 2(7 ft)(10 psf)
	=	478 plf
Roof snow	=	1/2(28 ft)(16 psf) = 224 plf
Live load (two floors)	=	(30 psf + 30 psf)(7 ft) = 420 plf
Attic live load	=	(10 psf)(14 ft - 5 ft*) = 90 plf *edges of roof span not accessible to roof storage due to low clearance

- Apply applicable ASD load combinations (Table 3.1)

(a) D + L + 0.3 (L_r or S)

Wall axial gravity load = 478 plf + 420 plf + 0.3 (224 plf)
= 965 plf*

*equals 1,055 plf if full attic live load allowance is included with L

(b) D + (L_r or S) + 0.3L

Wall axial gravity load = 478 plf + 224 plf + 0.3 (420 plf)
= 828 plf

Load condition (a) controls the gravity load analysis for the bearing wall. The same load applies to the design of headers as well as to the wall studs. Of course, combined lateral (bending) and axial loads on the wall studs also need to be checked (i.e., D+W); refer to Table 3.1 and Example 3.2. For nonload-bearing exterior walls (i.e., gable-end curtain walls), contributions from floor and roof live loads may be negligible (or significantly reduced), and the D+W load combination likely governs the design.

2. Gravity load on a column supporting a center floor girder carrying loads from two floors (first and second stories)

- Assume a column spacing of 16 ft
- Determine loads on column

(a) Dead load = Second floor + first floor + bearing wall supporting second floor
= (14 ft)(16 ft)(10 psf) + (14 ft)(16 ft)(10 psf) + (8 ft)(16 ft)(7 psf)
= 5,376 lbs

(b) Live load area reduction (Equation 3.4-1)

- supported floor area = 2(14 ft)(16 ft) = 448 ft^2 per floor
- reduction = $\left[0.25 + \frac{10.6}{\sqrt{448}}\right] = 0.75 \geq 0.75$ OK
- first-floor live load = 0.75 (40 psf) = 30 psf
- second-floor live load = 0.75 (30 psf) = 22.5 psf

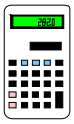
(c) Live load = (14 ft)(16 ft)[30 psf + 22.5 psf]
= 11,760 lbs

- Apply ASD load combinations (Table 3.1)

The controlling load combination is D+L since there are no attic or roof loads supported by the column. The total axial gravity design load on the column is 17,136 lbs (5,376 lbs + 11,760 lbs).

Note. If LRFD material design specifications are used, the various loads would be factored in accordance with Table 3.1. All other considerations and calculations remain unchanged.

EXAMPLE 3.2 — *Design Wind Load Calculations and Use of ASD Load Combinations*



Given

- Site wind speed–100 mph, gust
- Site wind exposure–suburban
- Two-story home, 7:12 roof pitch, 28' x 44' plan (rectangular), gable roof, 12-inch overhang

Find

1. Lateral (shear) load on lower-story end wall
2. Net roof uplift at connections to the side wall
3. Roof sheathing pull-off (suction) pressure
4. Wind load on a roof truss
5. Wind load on a rafter
6. Lateral (out-of-plane) wind load on a wall stud

Solution

1. Lateral (shear) load on lower-story end wall

Step 1: Velocity pressure = 14.6 psf (Table 3.7)
Step 2: Adjusted velocity pressure = 0.9* x 14.6 psf = 13.1 psf
*adjustment for wind directionality (V<110 mph)
Step 3: Lateral roof coefficient = 0.6 (Table 3.8)
Lateral wall coefficient = 1.2 (Table 3.8)
Step 4: Skip
Step 5: Determine design wind pressures
Wall projected area pressure = (13.1 psf)(1.2) = 15.7 psf
Roof projected area pressure = (13.1 psf)(0.6) = 7.9 psf

Now determine vertical projected areas (VPA) for lower-story end-wall tributary loading (assuming no contribution from interior walls in resisting lateral loads)

Roof VPA = [1/2 (building width)(roof pitch)] x [1/2 (building length)]
= [1/2 (28 ft)(7/12)] x [1/2 (44 ft)]
= [8.2 ft] x [22 ft]
= 180 ft^2

Wall VPA = [(second-story wall height) + (thickness of floor) + 1/2 (first-story wall height)] x [1/2 (building length)]
= [8 ft + 1 ft + 4 ft] x [1/2 (44 ft)]
= [13 ft] x [22 ft]
= 286 ft^2

Now determine shear load on the first-story end wall

Shear = (roof VPA)(roof projected area pressure) + (wall VPA)(wall projected area pressure)
= (180 ft^2)(7.9 psf) + (286 ft^2)(15.7 psf)
= 5,912 lbs

The first-story end wall must be designed to transfer a shear load of 5,169 lbs. If side-wall loads were determined instead, the vertical projected area would include only the gable-end wall area and the triangular wall area formed by the roof. Use of a hip roof would reduce the shear load for the side and end walls.

2. Roof uplift at connection to the side wall (parallel-to-ridge)

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Roof uplift pressure coefficient = -1.0 (Table 3.9)
Roof overhang pressure coefficient = 0.8 (Table 3.9)
Step 5: Determine design wind pressure
Roof horizontal projected area (HPA) pressure = -1.0 (13.1 psf)
= -13.1 psf
Roof overhang pressure = 0.8 (13.1 psf) = 10.5 psf (upward)

Now determine gross uplift at roof-wall reaction

Gross uplift = 1/2 (roof span)(roof HPA pressure) + (overhang)(overhang pressure coefficient)
= 1/2 (30 ft)(-13.1 psf) + (1 ft)(-10.5 psf)
= -207 plf (upward)

Roof dead load reaction = 1/2 (roof span)(uniform dead load)
= 1/2 (30 ft)(15 psf*)
*Table 3.2
= 225 plf (downward)

Now determine net design uplift load at roof-wall connection

Net design uplift load = $0.6D + W_u$ (Table 3.1)
= 0.6 (225 plf) + (-207 plf)
= -54 plf (net uplift)

The roof-wall connection must be capable of resisting a design uplift load of 54 plf. Generally, a toenail connection can be shown to meet the design requirement depending on the nail type, nail size, number of nails, and density of wall framing lumber (see Chapter 7). At appreciably higher design wind speeds or in more open wind exposure conditions, roof tie-down straps, brackets, or other connectors should be considered and may be required.

3. Roof sheathing pull-off (suction) pressure

Step 1: Velocity pressure	= 14.6 psf (as before)
Step 2: Adjusted velocity pressure	= 13.1 psf (as before)
Step 3: Skip	
Step 4: Roof sheathing pressure coefficient (suction)	= -2.2 (Table 3.9)
Step 5: Roof sheathing pressure (suction)	= (13.1 psf)(-2.2)
	= -28.8 psf

The fastener load depends on the spacing of roof framing and spacing of the fastener. Fasteners in the interior of the roof sheathing panel usually have the largest tributary area and therefore are critical. Assuming 24-inch-on-center roof framing, the fastener withdrawal load for a 12-inch-on-center fastener spacing is as follows:

Fastener withdrawal load = (fastener spacing)(framing spacing)
(roof sheathing pressure)
= (1 ft)(2 ft)(-28.8 psf)
= -57.6 lbs

This load exceeds the allowable capacity of minimum conventional roof sheathing connections (i.e., 6d nail). Therefore, a larger nail (i.e., 8d) would be required for the given wind condition. At appreciably higher wind conditions, a closer fastener spacing or higher-capacity fastener (i.e., deformed shank nail) may be required; refer to Chapter 7.

4. Load on a roof truss

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Roof truss pressure coefficient = -0.9, +0.4 (Table 3.9)
Step 5: Determine design wind pressures

(a) Uplift = -0.9 (13.1 psf) = -11.8 psf
(b) Inward = 0.4 (13.1 psf) = 5.2 psf

Since the inward wind pressure is less than the minimum roof live load (i.e., 15 psf, Table 3.4), the following load combinations would govern the roof truss design while the D+W load combination could be dismissed (refer to Table 3.1):

$D + (L_r \text{ or } S)$
$0.6D + W_u$*
*The net uplift load for truss design is relatively small in this case (approximately 3.5 psf) and may be dismissed by an experienced designer.

5. Load on a rafter

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Rafter pressure coefficient = -1.2, +0.7 (Table 3.9)
Step 5: Determine design wind pressures

(a) Uplift = (-1.2)(13.1 psf) = -15.7 psf
(b) Inward = (0.7)(13.1 psf) = 9.2 psf

Rafters in cathedral ceilings are sloped, simply supported beams, whereas rafters that are framed with cross-ties (i.e., ceiling joists) constitute a component (i.e., top chord) of a site-built truss system. Assuming the former in this case, the rafter should be designed as a sloped beam by using the span measured along the slope. By inspection, the minimum roof live load ($D+L_r$) governs the design of the rafter in comparison to the wind load combinations (see Table 3.1). The load combination 0.6 $D+W_u$ can be dismissed in this case for rafter sizing but must be considered when investigating wind uplift for the rafter-to-wall and rafter-to-ridge beam connections.

6. Lateral (out-of-plane) wind load on a wall stud

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Wall stud pressure coefficient = -1.2, +1.1 (Table 3.9)
Step 5: Determine design wind pressures

(a) Outward = (-1.2)(13.1 psf) = -15.7 psf
(b) Inward = (1.1)(13.1 psf) = 14.4 psf

Obviously, the outward pressure of 15.7 psf governs the out-of-plane bending load design of the wall stud. Since the load is a lateral pressure (not uplift), the applicable load combination is D+W (refer to Table 3.1), resulting in a combined axial and bending load. The axial load would include the tributary building dead load from supported assemblies (i.e., walls, floors, and roof). The bending load would the be determined by using the wind pressure of 15.7 psf applied to the stud as a uniform line load on a simply supported beam calculated as follows:

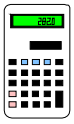
Uniform line load, w = (wind pressure)(stud spacing)
= (15.7 psf)(1.33 ft*)
*assumes a stud spacing of 16 inches on center
= 20.9 plf

Of course, the following gravity load combinations would also need to be considered in the stud design (refer to Table 3.1):

$D + L + 0.3 (L_r \text{ or } S)$
$D + (L_r \text{ or } S) + 0.3 L$

It should be noted that the stud is actually part of a wall system (i.e., sheathing and interior finish) and can add substantially to the calculated bending capacity; refer to Chapter 5.

EXAMPLE 3.3 Design Earthquake Load Calculation



Given

- Site ground motion, S_s = 1g
- Site soil condition = firm (default)
- Roof snow load < 30 psf
- Two-story home, 28' x 44' plan, typical construction

Find Design seismic shear on first-story end wall assuming no interior shear walls or contribution from partition walls

Solution

1. Determine tributary mass (weight) of building to first-story seismic shear

Roof dead load = (28 ft)(44 ft)(15 psf) = 18,480 lb
Second-story exterior wall dead load = (144 lf)(8 ft)(8 psf) = 9,216 lb
Second-story partition wall dead load = (28 ft)(44 ft)(6 psf) = 7,392 lb
Second-story floor dead load = (28 ft)(44 ft)(10 psf) = 12,320 lb
First-story exterior walls (1/2 height) = (144 lf)(4 ft)(8 psf) = 4,608 lb
Assume first-story interior partition walls are capable of at least supporting the seismic shear produced by their own weight

Total tributary weight = 52,016 lb

2. Determine total seismic story shear on first story

$S_{DS} = 2/3\ (S_s)(F_a)$ (Equation 3.8-2)
$= 2/3\ (1.0g)(1.1)$ (F_a = 1.1 from Table 3.11)
$= 0.74\ g$

$$V = \frac{1.2\ S_{DS}}{R} W$$

$$= \frac{1.2\,(0.74g)}{5.5}(52{,}016\ \text{lb})$$ (R = 5.5 from Table 3.12)

$= 8{,}399$ lb

3. Determine design shear load on the 28-foot end walls

Assume that the building mass is evenly distributed and that stiffness is also reasonably balanced between the two end walls; refer to Chapter 6 for additional guidance.

With the above assumption, the load is simply distributed to the end walls according to tributary weight (or plan area) of the building. Therefore,

End wall shear = 1/2 (8,399 lb) = 4,200 lb

Note that the design shear load from wind (100 mph gust, exposure B) in Example 3.2 is somewhat greater (5,912 lbs).

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CHAPTER 4

Design of Foundations

4.1 General

A foundation transfers the load of a structure to the earth and resists loads imposed by the earth. A foundation in residential construction may consist of a footing, wall, slab, pier, pile, or a combination of these elements. This chapter addresses the following foundation types:

- crawl space;
- basement;
- slab-on-grade with stem wall;
- monolithic slab;
- piles;
- piers; and
- alternative methods.

As discussed in Chapter 1, the most common residential foundation materials are concrete masonry (i.e., concrete block) and cast-in-place concrete. Preservative-treated wood, precast concrete, and other methods may also be used. The concrete slab on grade is the most popular foundation type in the Southeast; basements are the most common type in the East and Midwest. Crawl spaces are common in the Northwest and Southeast. Pile foundations are commonly used in coastal flood zones to elevate structures above flood levels, in weak or expansive soils to reach a stable stratum, and on steeply sloped sites. Figure 4.1 depicts different foundation types; a brief description follows.

A *crawl space* is a building foundation that uses a perimeter foundation wall to create an under-floor space that is not habitable; the interior crawl space elevation may or may not be below the exterior finish grade. A *basement* is typically defined as a portion of a building that is partly or completely below the exterior grade and that may be used as habitable or storage space.

A *slab on grade with an independent stem wall* is a concrete floor supported by the soil independently of the rest of the building. The stem wall supports the building loads and in turn is supported directly by the soil or a

footing. A *monolithic or thickened-edge slab* is a ground-supported slab on grade with an integral footing (i.e., thickened edge); it is normally used in warmer regions with little or no frost depth but is also used in colder climates when adequate frost protection is provided (see Section 4.7).

When necessary, *piles* are used to transmit the load to a deeper soil stratum with a higher bearing capacity, to prevent failure due to undercutting of the foundation by scour from flood water flow at high velocities, and to elevate the building above required flood elevations. Piles are also used to isolate the structure from expansive soil movements.

Post-and-pier foundations can provide an economical alternative to crawl space perimeter wall construction. It is common practice to use a brick curtain wall between piers for appearance and bracing purposes.

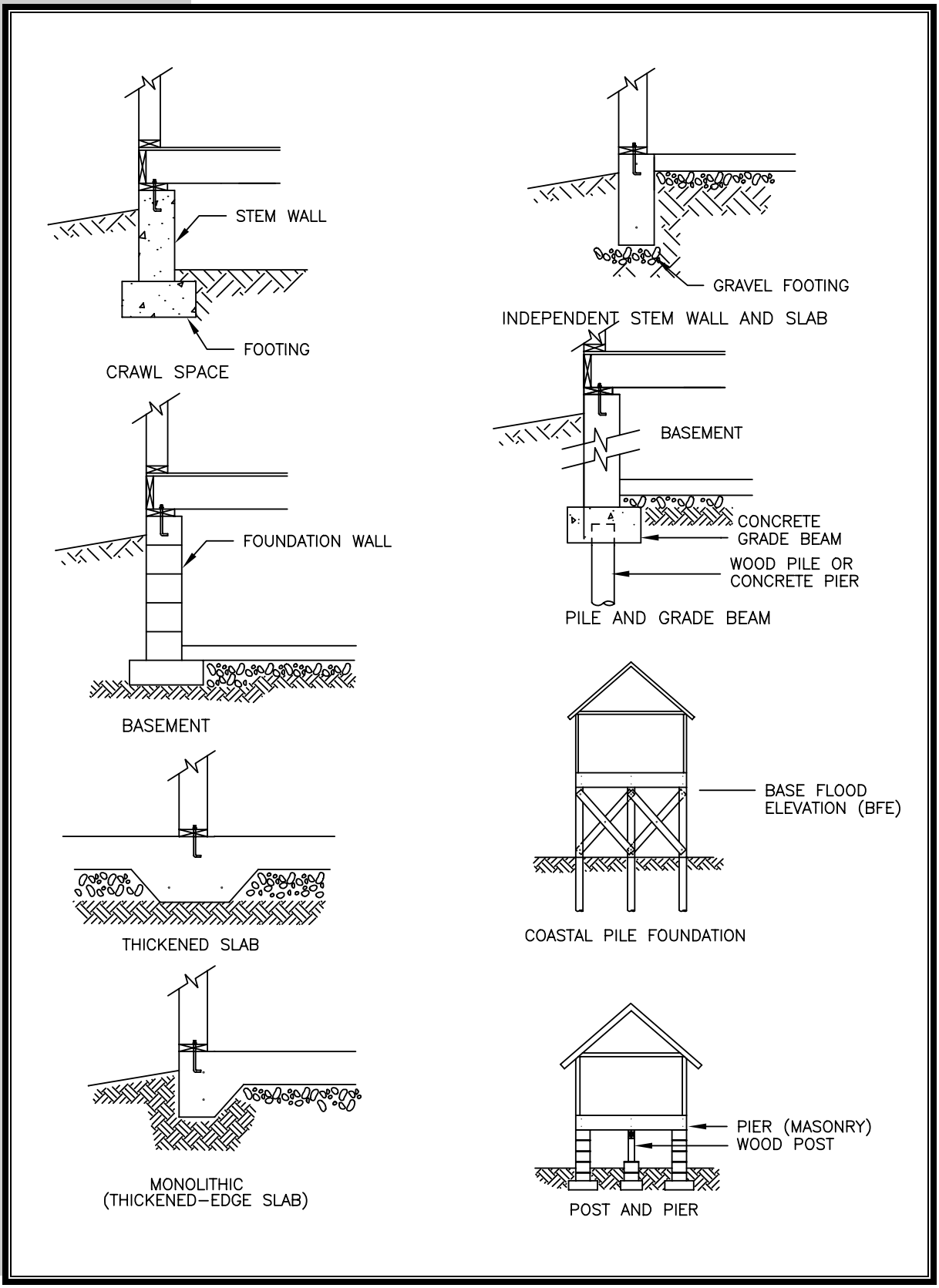
The design procedures and information in this chapter cover

- foundation materials and properties;
- soil bearing capacity and footing size;
- concrete or gravel footings;
- concrete and masonry foundation walls;
- preservative-treated wood walls;
- insulating concrete foundations;
- concrete slabs on grade;
- pile foundations; and
- frost protection.

Concrete design procedures generally follow the strength design method contained in ACI-318 (ACI, 1999), although certain aspects of the procedures may be considered conservative relative to conventional residential foundation applications. For this reason, some supplemental design guidance is provided when practical and technically justified. Masonry design procedures follow the allowable stress design method of ACI-530 (ACI, 1999). Wood design procedures are used to design the connections between the foundation system and the structure above and follow the allowable stress design method for wood construction; refer to Chapter 7 for connection design information. In addition, the designer is referred to the applicable design standards for symbol definitions and additional guidance since the intent of this chapter is to provide supplemental instruction in the efficient design of residential foundations.

As a matter of consistency within the scope of this guide, the LRFD load combinations of Chapter 3 (Table 3.1) are used in lieu of those required in ACI-318 for strength design of concrete. The designer is advised of this variance from what may be considered accepted practice in the local building code. However, the intent is to provide designs that are at least consistent with current residential building code and construction practice. With respect to the design of concrete in residential foundations, it is also intended to provide reasonable safety margins that are at least consistent with the minimums required for other more crucial (i.e., life-safety) elements of a home. If an actual design is performed in accordance with this guide, it is the responsibility of the designer to seek any special approval that may be required for “alternative means and methods” of design and to identify where and when such approval is needed.

FIGURE 4.1 Types of Foundations



4.2 Material Properties

A residential designer using concrete and masonry materials must have a basic understanding of such materials as well as an appreciation of variations in the materials' composition and structural properties. In addition, soils are considered a foundation material (Section 4.3 provides information on soil bearing). A brief discussion of the properties of concrete and masonry follows.

4.2.1 Concrete

The concrete compressive strength f_c' used in residential construction is typically either 2,500 or 3,000 psi, although other values may be specified. For example, 3,500 psi concrete may be used for improved weathering resistance in particularly severe climates or unusual applications. The concrete compressive strength may be verified in accordance with ASTM C39 (ASTM, 1996). Given that concrete strength increases at a diminishing rate with time, the specified compressive strength is usually associated with the strength attained after 28 days of curing time. At that time, concrete generally attains about 85 percent of its fully cured compressive strength.

Concrete is a mixture of cement, water, sand, gravel, crushed rock, or other aggregates. Sometimes one or more admixtures are added to change certain characteristics of the concrete, such as workability, durability, and time of hardening. The proportions of the components determine the concrete mix's compressive strength and durability.

Type

Portland cement is classified into several types in accordance with ASTM C150 (ASTM, 1998). Residential foundation walls are typically constructed with *Type I* cement, which is a general-purpose Portland cement used for the vast majority of construction projects. Other types of cement are appropriate in accommodating conditions related to heat of hydration in massive pours and sulfate resistance. In some regions, sulfates in soils have caused durability problems with concrete. The designer should check into local conditions and practices.

Weight

The weight of concrete varies depending on the type of aggregates used in the concrete mix. Concrete is typically referred to as lightweight or normal weight. The density of unreinforced normal weight concrete ranges between 144 and 156 pounds per cubic foot (pcf) and is typically assumed to be 150 pcf. Residential foundations are constructed with normal weight concrete.

Slump

Slump is the measure of concrete consistency; the higher the slump, the wetter the concrete and the easier it flows. Slump is measured in accordance with ASTM C143 (ASTM, 1998) by inverting a standard 12-inch-high metal cone, filling it with concrete, and then removing the cone; the amount the concrete settles in units of inches is the slump. Most foundations, slabs, and walls consolidated by hand methods have a slump between 4 and 6 inches. One problem associated with a high-slump concrete is segregation of the aggregate, which leads to cracking and scaling. Therefore, a slump of greater than 6 should be avoided.

Admixtures

Admixtures are materials added to the concrete mix to improve workability and durability and to retard or accelerate curing. Some of the most common admixtures are described below.

- *Water reducers* improve the workability of concrete without reducing its strength.
- *Retarders* are used in hot weather to allow more time for placing and finishing concrete. Retarders may also reduce the early strength of concrete.
- *Accelerators* reduce the setting time, allowing less time for placing and finishing concrete. Accelerators may also increase the early strength of concrete.
- *Air-entrainers* are used for concrete that will be exposed to freeze-thaw conditions and deicing salts. Less water is needed, and desegregation of aggregate is reduced when air-entrainers are added.

Reinforcement

Concrete has high compressive strength but low tensile strength; therefore, reinforcing steel is often embedded in the concrete to provide additional tensile strength and ductility. In the rare event that the capacity may be exceeded, the reinforcing steel begins to yield, eliminating an abrupt failure that may otherwise occur in plain, unreinforced concrete. For this reason, a larger safety margin is used in the design of plain concrete construction than in reinforced concrete construction.

Steel reinforcement is available in Grade 40 or Grade 60; the grade number refers to the minimum tensile yield strength f_y of the steel (i.e., Grade 40 is minimum 40 ksi steel and Grade 60 is minimum 60 ksi steel). Either grade may be used for residential construction; however, most reinforcement in the U.S. market today is Grade 60. It is also important that the concrete mix or slump is adjusted through the addition of an appropriate amount of water to allow the concrete to flow easily around the reinforcement bars, particularly when the bars are closely spaced or crowed at points of overlap. However, close spacing is rarely required in residential construction and should be avoided in design.

The most common steel reinforcement or rebar sizes in residential construction are No. 3, No. 4, and No. 5, which correspond to diameters of 3/8-inch, 1/2-inch, and 5/8-inch, respectively. These three sizes of rebar are easily handled at the jobsite by using manual bending and cutting devices. Table 4.1 provides useful relationships among the rebar number, diameter, and cross-sectional for reinforced concrete and masonry design.

TABLE 4.1 Rebar Size, Diameter, and Cross-Sectional Areas

Size	Diameter (inches)	Area (square inches)
No. 3	3/8	0.11
No. 4	1/2	0.20
No. 5	5/8	0.31
No. 6	3/4	0.44
No. 7	7/8	0.60
No. 8	1	0.79

4.2.2 Concrete Masonry Units

Concrete masonry units (CMU) are commonly referred to as concrete blocks. They are composed of Portland cement, aggregate, and water. Admixtures may also be added in some situations. Low-slump concrete is molded and cured to produce strong blocks or units. Residential foundation walls are typically constructed with units 7-5/8 inches high by 15-5/8 inches long, providing a 3/8-inch allowance for the width of mortar joints.

In residential construction, nominal 8-inch-thick concrete masonry units are readily available. It is generally more economical if the masonry unit compressive strength f'_m ranges between 1,500 and 3,000 psi. The standard block used in residential and light-frame commercial construction is generally rated with a design strength f'_m of 1,900 psi, although other strengths are available.

Grade

Concrete masonry units are described by grades according to their intended use per ASTM C90 (ASTM, 1999) or C129 (ASTM, 1999). Residential foundation walls should be constructed with *Grade N* units. *Grade S* may be used above grade. The grades are described below.

- *Grade N* is typically required for general use such as in interior and backup walls and in above- or below-grade exterior walls that may or may not be exposed to moisture penetration or the weather.
- *Grade S* is typically limited to above-grade use in exterior walls with weather-protective coatings and in walls not exposed to the weather.

Type

Concrete masonry units are classified in accordance with ASTM C90 as *Type I or II* (ASTM, 1999). *Type I* is a moisture-controlled unit that is typically specified where drying shrinkage of the block due to moisture loss may result in excessive cracking in the walls. *Type II* is a nonmoisture-controlled unit that is suitable for all other uses. Residential foundation walls are typically constructed with *Type II* units.

Weight

Concrete masonry units are available with different densities by altering the type(s) of aggregate used in their manufacture. Concrete masonry units are typically referred to as lightweight, medium weight, or normal weight with respective unit weights or densities less than 105 pcf, between 105 and 125 pcf, and more than 125 pcf. Residential foundation walls are typically constructed with low- to medium-weight units because of the low compressive strength required. However, lower-density units are generally more porous and must be properly protected to resist moisture intrusion. A common practice in residential basement foundation wall construction is to provide a cement-based parge coating and a brush- or spray-applied bituminous coating on the below-ground portions of the wall. This treatment is usually required by code for basement walls of masonry or concrete construction; however, in concrete construction, the parge coating is not necessary.

Hollow or Solid

Concrete masonry units are classified as hollow or solid in accordance with ASTM C90 (ASTM, 1999). The net concrete cross-sectional area of most concrete masonry units ranges from 50 to 70 percent depending on unit width, face-shell and web thicknesses, and core configuration. *Hollow* units are defined as those in which the net concrete cross-sectional area is less than 75 percent of the gross cross-sectional area. *Solid* units are not necessarily solid but are defined as those in which the net concrete cross-sectional area is 75 percent of the gross cross-sectional area or greater.

Mortar

Masonry mortar is used to join concrete masonry units into a structural wall; it also retards air and moisture infiltration. The most common way to lay block is in a running bond pattern where the vertical head joints between blocks are offset by half the block length from one course to the next. Mortar is composed of cement, lime, clean, well-graded sand, and water and is typically classified into *Types M, S, N, O,* and *K* in accordance with ASTM C270 (ASTM, 1999). Residential foundation walls are typically constructed with *Type M* or *Type S* mortar, both of which are generally recommended for load-bearing interior and exterior walls including above- and below-grade applications.

Grout

Grout is a slurry consisting of cementitious material, aggregate, and water. When needed, grout is commonly placed in the hollow cores of concrete masonry units to provide a wall with added strength. In reinforced load-bearing masonry wall construction, grout is usually placed only in those hollow cores containing steel reinforcement. The grout bonds the masonry units and steel so that they act as a composite unit to resist imposed loads. Grout may also be used in unreinforced concrete masonry walls for added strength.

4.3 Soil Bearing Capacity and Footing Size

Soil bearing investigations are rarely required for residential construction except in the case of known risks as evidenced by a history of local problems (e.g., organic deposits, landfills, expansive soils, etc.). Soil bearing tests on stronger-than-average soils can, however, justify smaller footings or eliminate footings entirely if the foundation wall provides sufficient bearing surface. For a conservative relationship between soil type and load-bearing value, refer to Table 4.2. A similar table is typically published in the building codes.

TABLE 4.2 Presumptive Soil Bearing Values by Soil Description

Presumptive Load-Bearing Value (psf)	Soil Description
1,500	Clay, sandy clay, silty clay, clayey silt, silt, and sandy silt
2,000	Sand, silty sand, clayey sand, silty gravel, and clayey gravel
3,000	Gravel and sandy gravel
4,000	Sedimentary rock
12,000	Crystalline bedrock

Source: Naval Facilities Command, 1986.

When a soil bearing investigation is desired to determine more accurate and economical footing requirements, the designer commonly turns to ASTM D1586, *Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils* (ASTM, 1999). This test relies on a 2-inch-diameter device driven into the ground with a 140-pound hammer dropped from a distance of 30 inches. The number of hammer drops or blows needed to create a one-foot penetration (blow count) is recorded. Values can be roughly correlated to soil bearing values as shown in Table 4.3. The instrumentation and cost of conducting the SPT test is usually not warranted for typical residential applications. Nonetheless, the SPT test method provides information on deeper soil strata and thus can offer valuable guidance for foundation design and building location, particularly when subsurface conditions are suspected to be problematic. The values in Table 4.3 are associated

with the blow count from the SPT test method. Many engineers can provide reasonable estimates of soil bearing by using smaller penetrometers at less cost, although such devices and methods may require an independent calibration to determine presumptive soil bearing values and may not be able to detect deep subsurface problems. Calibrations may be provided by the manufacturer or, alternatively, developed by the engineer.

The designer should exercise judgment when selecting the final design value and be prepared to make adjustments (increases or decreases) in interpreting and applying the results to a specific design. The values in Tables 4.2 and 4.3 are generally associated with a safety factor of 3 (Naval Facilities Engineering Command, 1996) and are considered appropriate for noncontinuous or independent spread footings supporting columns or piers (i.e., point loads). Use of a minimum safety factor of 2 (corresponding to a higher presumptive soil bearing value) is recommended for smaller structures with continuous spread footings such as houses. To achieve a safety factor of 2, the designer may multiply the values in Tables 4.2 and 4.3 by 1.5.

Table 4.3 Presumptive Soil Bearing Values (psf) Based on Standard Penetrometer Blow Count

In Situ Consistency, N[1]		Loose[2] (5 to 10 blows per foot)	Firm (10 to 25 blows per foot)	Compact (25 to 50 blows per foot)
Noncohesive Soils	Gravel	4,000 (10)	8,000 (25)	11,000 (50)
	Sand	2,500 (6)	5,000 (20)	6,000 (35)
	Fine sand	1,000 (5)	3,000 (12)	5,000 (30)
	Silt	500 (5)	2,000 (15)	4,000 (35)
Insitu Consistency, N[1]:		**Soft[3] (3 to 5 blows per foot)**	**Medium (about 10 blows per foot)**	**Stiff (> 20 blows per foot)**
Cohesive Soils	Clay, Sand, Gravel Mixtures	2,000 (3)	5,000 (10)	8,000 (20)
	Sandy or Silty Clay	1,000 (4)	3,000 (8)	6,000 (20)
	Clay	500 (5)	2,000 (10)	4,000 (25)

Source: Naval Facilities Command, 1986.

Notes:

[1]N denotes the standard penetrometer blow count in blows per foot in accordance with ASTM D1586; shown in parentheses.

[2]Compaction should be considered in these conditions, particularly when the blow count is five blows per foot or less.

[3]Pile and grade beam foundations should be considered in these conditions, particularly when the blow count is five blows per foot or less.

The required width or area of a spread footing is determined by dividing the building load on the footing by the soil bearing capacity from Table 4.2 or Table 4.3 as shown below. Building design loads, including dead and live loads, should be determined in accordance with Chapter 3 by using allowable stress design (ASD) load combinations.

$$\text{Area}_{\text{independent spread footing}} = \frac{\text{Load in lbs}}{\text{Soil bearing capacity in psf}}$$

$$\text{Width}_{\text{continuous footing}} = \frac{\text{Load in plf}}{\text{Soil bearing capacity in psf}}$$

4.4 Footings

The objectives of footing design are

- to provide a level surface for construction of the foundation wall;
- to provide adequate transfer and distribution of building loads to the underlying soil;
- to provide adequate strength, in addition to the foundation wall, to prevent differential settlement of the building in weak or uncertain soil conditions;
- to place the building foundation at a sufficient depth to avoid frost heave or thaw weakening in frost-susceptible soils and to avoid organic surface soil layers; and
- to provide adequate anchorage or mass (when needed in addition to the foundation wall) to resist potential uplift and overturning forces resulting from high winds or severe seismic events.

This section presents design methods for concrete and gravel footings. The designer is reminded that the required footing width is first established in accordance with Section 4.3. Further, if soil conditions are stable or the foundation wall can adequately resist potential differential settlement, the footing may be completely eliminated.

By far, the most common footing in residential construction is a continuous concrete spread footing. However concrete and gravel footings are both recognized in prescriptive footing size tables in residential building codes for most typical conditions (ICC, 1998). In contrast, special conditions give rise to some engineering concerns that need to be addressed to ensure the adequacy of any foundation design. Special conditions include

- steeply sloped sites requiring a stepped footing;
- high-wind conditions;
- inland or coastal flooding conditions;
- high-hazard seismic conditions; and
- poor soil conditions.

4.4.1 Simple Gravel and Concrete Footing Design

Building codes for residential construction contain tables that prescribe minimum footing widths for plain concrete footings (ICC, 1998). Alternatively, footing widths may be determined in accordance with Section 4.3 based on a

site's particular loading condition and presumptive soil bearing capacity. The following are general rules of thumb for determining the thickness of plain concrete footings for residential structures once the required bearing width is calculated:

- The minimum footing thickness should not be less than the distance the footing extends outward from the edge of the foundation wall or 6 inches, whichever is greater.
- The footing width should project a minimum of 2 inches from both faces of the wall (to allow for a minimum construction tolerance) but not greater than the footing thickness.

These rules of thumb generally result in a footing design that differs somewhat from the plain concrete design provisions of Chapter 22 of ACI-318. It should also be understood that footing widths generally follow the width increments of standard excavation equipment (i.e., a backhoe bucket size of 12, 16, or 24 inches). Even though some designers and builders may specify one or two longitudinal No. 4 bars for wall footings, steel reinforcement is not required for residential-scale structures in typical soil conditions. For situations where the rules of thumb or prescriptive code tables do not apply or where a more economical solution is possible, a more detailed footing analysis may be considered (see Section 4.4.2). Refer to Example 4.1 for a plain concrete footing design in accordance with the simple method described herein.

Much like a concrete footing, a gravel footing may be used to distribute foundation loads to a sufficient soil bearing surface area. It also provides a continuous path for water or moisture and thus must be drained in accordance with the foundation drainage provisions of the national building codes. Gravel footings are constructed of crushed stone or gravel that is consolidated by tamping or vibrating. Pea gravel, which is naturally consolidated, does not require compaction and can be screeded to a smooth, level surface much like concrete. Although typically associated with pressure-treated wood foundations (refer to Section 4.5.3), a gravel footing can support cast-in-place or precast concrete foundation walls.

The size of a gravel footing is usually based on a 30- to 45-degree angle of repose for distributing loads; therefore, as with plain concrete footings, the required depth and width of the gravel footing depends on the width of the foundation wall, the foundation load, and soil bearing values. Following a rule of thumb similar to that for a concrete footing, the gravel footing thickness should be no less than 1.5 times its extension beyond the edge of the foundation wall or, in the case of a pressure-treated wood foundation, the mud sill. Just as with a concrete footing, the thickness of a gravel footing may be considered in meeting the required frost depth. In soils that are not naturally well-drained, provision should be made to adequately drain a gravel footing.

4.4.2 Concrete Footing Design

For the vast majority of residential footing designs, it quickly becomes evident that conventional residential footing requirements found in residential building codes are adequate, if not conservative (ICC,1998). However, to improve

performance and economy or to address peculiar conditions, a footing may need to be specially designed.

A footing is designed to resist the upward-acting pressure created by the soil beneath the footing; that pressure tends to make the footing bend upward at its edges. According to ACI-318, the three modes of failure considered in reinforced concrete footing design are one-way shear, two-way shear, and flexure (see Figure 4.2). Bearing (crushing) is also a possible failure mode, but is rarely applicable to residential loading conditions. To simplify calculations for the three failure modes, the following discussion explains the relation of the failure modes to the design of plain and reinforced concrete footings. The designer should refer to ACI-318 for additional commentary and guidance. The design equations used later in this section are based on ACI-318 and principles of engineering mechanics as described below. Moreover, the approach is based on the assumption of uniform soil bearing pressure on the bottom of the footing; therefore, walls and columns should be supported as close as possible to the center of the footings.

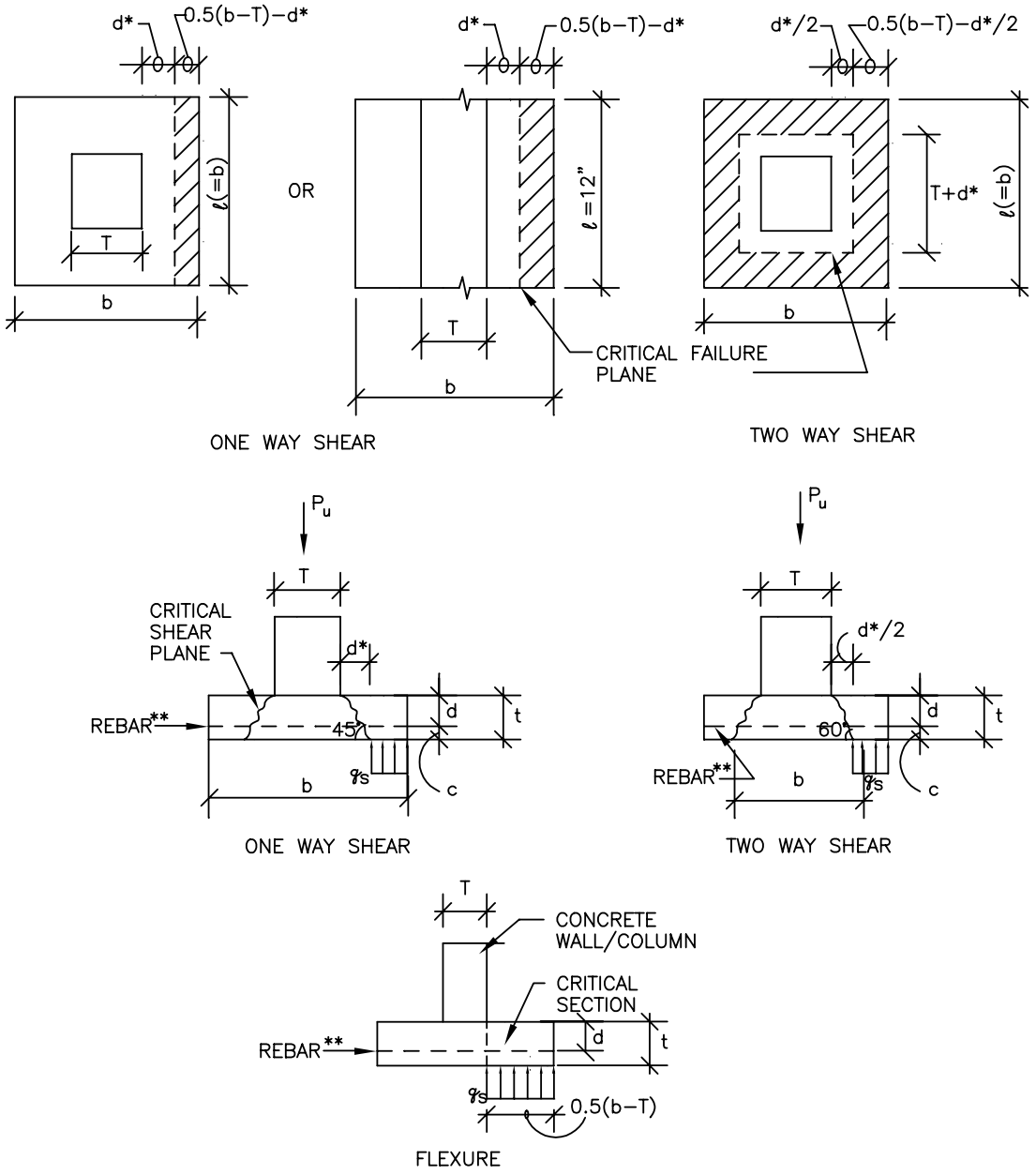
One-Way (Beam) Shear

When a footing fails due to *one-way (beam) shear*, the failure occurs at an angle approximately 45 degrees to the wall as shown in Figure 4.2. For plain concrete footings, the soil bearing pressure has a negligible effect on the diagonal shear tension for distance t from the wall edge toward the footing edge; for reinforced concrete footings, the distance used is d, which equals the depth to the footing rebar (see Figure 4.2). As a result, one-way shear is checked by assuming that beam action occurs at a critical failure plane extending across the footing width as shown in Figure 4.2. One-way shear must be considered in similar fashion in both continuous wall and rectangular footings; however, for ease of calculation, continuous wall footing design is typically based on one lineal foot of wall/footing.

Two-Way (Punching) Shear

When a footing fails by *two-way (punching) shear*, the failure occurs at an angle approximately 30 degrees to the column or pier as shown in Figure 4.2. Punching shear is rarely a concern in the design of continuous wall footings and thus is usually checked only in the case of rectangular or circular footings with a heavily loaded pier or column that creates a large concentrated load on a relatively small area of the footing. For plain concrete footings, the soil bearing pressure has a negligible effect on the diagonal shear tension at distance $t/2$ from the face of a column toward the footing edges; for reinforced concrete footings, the distance from the face of the column is $d/2$ (see Figure 4.2). Therefore, the shear force consists of the net upward-acting pressure on the area of the footing outside the “punched-out” area (hatched area in Figure 4.2). For square, circular, or rectangular footings, shear is checked at the critical section that extends in a plane around a concrete, masonry, wood, or steel column or pier that forms the perimeter b_o of the area described above.

FIGURE 4.2 *Critical Failure Planes in Continuous or Square Concrete Spread Footings*



NOTES: * SUBSTITUTE t FOR d AS REQUIRED FOR PLAIN CONCRETE FOOTING DESIGN

** REBAR IS REQUIRED ONLY IN REINFORCED CONCRETE FOOTING DESIGN AND IS SHOWN HERE FOR THAT PURPOSE ONLY. IN REINFORCED SQUARE FOOTINGS, THE REBAR MUST BE PLACED IN TWO DIRECTIONS.

Flexure (Bending)

The maximum moment in a footing deformed by the upward-acting soil pressures would logically occur in the middle of the footing; however, the rigidity of the wall or column above resists some of the upward-acting forces and affects the location of maximum moment. As a result, the critical flexure plane for footings supporting a rigid wall or column is assumed to be located at the face of the wall or column. Flexure in a concrete footing is checked by computing the moment created by the soil bearing forces acting over the cantilevered area of the footing that extends from the critical flexure plane to the edge of the footing (hatched area in Figure 4.2). The approach for masonry walls in ACI-318 differs slightly in that the failure plane is assumed to be located one-fourth of the way under a masonry wall or column, creating a slightly longer cantilever. For the purpose of this guide, the difference is considered unnecessary.

Bearing Strength

It is difficult to contemplate conditions where concrete bearing or compressive strength is a concern in typical residential construction; therefore, a design check can usually be dismissed as "OK by inspection." In rare and peculiar instances where bearing compressive forces on the concrete are extreme and approach or exceed the specified concrete compressive strength, ACI-318•10.17 and ACI-318•12.3 should be consulted for appropriate design guidance.

4.4.2.1 Plain Concrete Footing Design

In this section, the design of plain concrete footings is presented by using the concepts related to shear and bending covered in the previous section. Refer to Example 4.1 in Section 4.9 for a plain concrete footing design example.

Shear

In the equations given below for one- and two-way shear, the dimensions are in accordance with Figure 4.2; units of inches should be used. ACI-318 requires an additional 2 inches of footing thickness to compensate for uneven trench conditions and does not allow a total footing thickness less than 8 inches for plain concrete. These limits may be relaxed for residential footing design, provided that the capacity is shown to be sufficient in accordance with the ACI-318 design equations. Footings in residential construction are often 6 inches thick. The equations below are specifically tailored for footings supporting walls or square columns since such footings are common in residential construction. The equations may be generalized for use with other conditions (i.e., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. In addition, the terms $4/3\sqrt{f'_c}$ and $4\sqrt{f'_c}$ are in units of pounds per square inch and represent "lower-bound" estimates of the ultimate shear stress capacity of unreinforced concrete.

[ACI-318•22.5,22.7]

One-Way (Beam) Shear

$\phi V_c \geq V_u$	basic design check for shear
$V_u = (q_s)(0.5(b-T)-t)\ell$	factored shear load (lb)
$q_s = \frac{P_u}{b\ell}$	uniform soil bearing pressure (psi) due to factored foundation load P_u (lb)
$\phi V_c = \phi\frac{4}{3}\sqrt{f'_c}lt$	factored shear capacity (lb)
$\phi = 0.65$	resistance factor

Two-Way (Punching) Shear

$\phi V_c \geq V_u$	basic design check for shear
$V_u = (q_s)\left(b\ell-(T+t)^2\right)$	shear load (lb) due to factored load P_u (lb)
$q_s = \frac{P_u}{b\ell}$	uniform soil bearing pressure (psi) due to factored foundation load P_u (lb)
$\phi V_c = \phi 4\sqrt{f'_c}b_o t$	factored shear capacity (lb)
$b_o = 4(T+t)$	perimeter of critical failure plane around a square column or pier
$\phi = 0.65$	resistance factor

Flexure

For a plain concrete footing, flexure (bending) is checked by using the equations below for footings that support walls or square columns (see Figure 4.2). The dimensions in the equations are in accordance with Figure 4.2 and use units of inches. The term $5\sqrt{f'_c}$ is in units of pounds per square inch (psi) and represents a "lower-bound" estimate of the ultimate tensile (rupture) stress of unreinforced concrete in bending.

[ACI-318•22.5,22.7]

$\phi M_n \geq M_u$	basic design check for bending
$M_u = \frac{1}{8}q_s\ell(b-T)^2$	factored moment (in-lb) due to soil pressure q_s (psi) acting on cantilevered portion of footing
$q_s = \frac{P_u}{bl}$	uniform soil bearing pressure (psi) due to factored load P_u (lb)
$\phi M_n = \phi 5\sqrt{f'_c}S$	factored moment capacity (in-lb) for plain concrete
$S = \frac{1}{6}\ell t^2$	section modulus (in^3) for footing
$\phi = 0.65$	resistance factor for plain concrete in bending

4.4.2.2 Reinforced Concrete Footing Design

For infrequent situations in residential construction where a plain concrete footing may not be practical or where it is more economical to reduce the footing thickness, steel reinforcement may be considered. A reinforced concrete footing is designed similar to a plain concrete footing; however, the concrete depth *d* to the reinforcing bar is used to check shear instead of the entire footing thickness *t*. The depth of the rebar is equal to the thickness of the footing minus the diameter of the rebar d_b and the concrete cover *c*. In addition, the moment capacity is determined differently due to the presence of the reinforcement, which resists the tension stresses induced by the bending moment. Finally, a higher resistance factor is used to reflect the more consistent bending strength of reinforced concrete relative to unreinforced concrete.

As specified by ACI-318, a minimum of 3 inches of concrete cover over steel reinforcement is required when concrete is in contact with soil. In addition, ACI-318 does not permit a depth *d* less than 6 inches for reinforced footings supported by soil. These limits may be relaxed by the designer, provided that adequate capacity is demonstrated in the strength analysis; however, a reinforced footing thickness of significantly less than 6 inches may be considered impractical even though it may calculate acceptably. One exception may be found where a nominal 4-inch-thick slab is reinforced to serve as an integral footing for an interior load-bearing wall (that is not intended to transmit uplift forces from a shear wall overturning restraint anchorage in high-hazard wind or seismic regions). Further, the concrete cover should not be less than 2 inches for residential applications, although this recommendation may be somewhat conservative for interior footings that are generally less exposed to ground moisture and other corrosive agents. Example 4.2 of Section 4.9 illustrates reinforced concrete footing design.

Shear

In the equations given below for one- and two-way shear, the dimensions are in accordance with Figure 4.2; units of inches should be used. Shear reinforcement (i.e., stirrups) is usually considered impractical for residential footing construction; therefore, the concrete is designed to withstand the shear stress as expressed in the equations. The equations are specifically tailored for footings supporting walls or square columns since such footings are common in residential construction. The equations may be generalized for use with other conditions (i.e., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. In addition, the terms $2\sqrt{f'_c}$ and $4\sqrt{f'_c}$ are in units of pounds per square inch and represent "lower-bound" estimates of the ultimate shear stress capacity of reinforced concrete.

[ACI-318•11.12,15.5]

One-Way (Beam) Shear

$\phi V_c \geq V_u$	basic design check for shear
$V_u = (q_s)(0.5(b-T)-d)\ell$	shear load (lb) due to uniform soil bearing pressure, q_s (psi)
$q_s = \frac{P_u}{b\ell}$	uniform solid bearing pressure (psi) due to factored foundation load P_u (lb)
$\phi V_c = \phi 2\sqrt{f'_c}\,\ell d$	factored shear capacity (lb)
$d = t - c - 0.5d_b$	depth of reinforcement
$\phi = 0.85$	resistance factor for reinforced concrete in shear

Two-Way (Punching) Shear

$\phi V_c \geq V_u$	basic design check for shear
$V_u = \left(\frac{P_u}{b\ell}\right)\left(b\ell - (T+d)^2\right)$	shear load (lb) due to factored load P_u (lb)
$\phi V_c = \phi 4\sqrt{f'_c}\,b_o d$	factored shear capacity (lb)
$b_o = 4(T+d)$	perimeter of punching shear failure plane around a square column or pier
$\phi = 0.85$	resistance factor for reinforced concrete in shear

Flexure

The flexure equations below pertain specifically to reinforced concrete footings that support walls or square columns. The equations may be generalized for use with other conditions (i.e., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. The alternative equation for nominal moment strength M_n is derived from force and moment equilibrium principles by using the provisions of ACI-318. Most designers are familiar with the alternative equation that uses the reinforcement ratio ρ and the nominal strength coefficient of resistance R_n. The coefficient is derived from the design check that ensures that the factored moment (due to factored loads) M_u is less than the factored nominal moment strength ϕM_n of the reinforced concrete. To aid the designer in short-cutting these calculations, design manuals provide design tables that correlate the nominal strength coefficient of resistance R_n to the reinforcement ratio ρ for a specific concrete compressive strength and steel yield strength.

[ACI-318•15.4]

$\phi M_n \geq M_u$ basic design check for bending

$M_u = \frac{1}{8} q_s \ell (b - T)^2$ factored moment (in-lb) due to soil pressure q_s (psi) acting on cantilevered portion of the footing

$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$ factored nominal moment capacity (in-lb)

$a = \frac{A_s f_y}{0.85 f'_c \ell}$ (l is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)

$\phi = 0.9$ resistance factor for reinforced concrete in bending

Alternate method to determine M_n

$\phi M_n = \phi \rho b d f_y \left(d - \frac{0.5 \rho d f_y}{0.85 f'_c} \right)$

$\rho = \left(\frac{0.85 f'_c}{f_y} \right) \left(\ell - \sqrt{\frac{2 R_n}{0.85 f'_c}} \right)$ reinforcement ratio determined by use of R_n

nominal strength "coefficient of resistance"

$R_n = \frac{M_u}{\phi \ell d^2}$ (l is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)

defines reinforcement ratio ρ

$A_s = \rho \ell d$ (l is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)

Minimum Reinforcement

Owing to concerns with shrinkage and temperature cracking, ACI-318 requires a minimum amount of steel reinforcement. The following equations determine minimum reinforcement, although many plain concrete residential footings have performed successfully and are commonly used. Thus, the ACI minimums may be considered arbitrary, and the designer may use discretion in applying the ACI minimums in residential footing design. The minimums certainly should not be considered a strict "pass/fail" criterion.

[ACI-318•7.12, 10.5]

$\rho_{min} = 200 / f_y$ or 0.0018

$A_{s,min} = \rho_{min} l d$ (l is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)

Designers often specify one or two longitudinal No. 4 bars for wall footings as nominal reinforcement in the case of questionable soils or when required to maintain continuity of stepped footings on sloped sites or under conditions resulting in a changed footing depth. However, for most residential foundations, the primary resistance against differential settlement is provided by the deep beam action of the foundation wall; footing reinforcement may provide

limited benefit. In such cases, the footing simply acts as a platform for the wall construction and distributes loads to a larger soil bearing area.

Lap Splices

Where reinforcement cannot be installed in one length to meet reinforcement requirements, as in continuous wall footings, reinforcement bars must be lapped to develop the bars' full tensile capacity across the splice. In accordance with ACI-318, a minimum lap length of 40 times the diameter of the reinforcement bar is required for splices in the reinforcement. In addition, the separation between spliced or lapped bars is not to exceed eight times the diameter of the reinforcement bar or 6 inches, whichever is less.

4.5 Foundation Walls

The objectives of foundation wall design are

- to transfer the load of the building to the footing or directly to the earth;
- to provide adequate strength, in combination with the footing when required, to prevent differential settlement;
- to provide adequate resistance to shear and bending stresses resulting from lateral soil pressure;
- to provide anchorage for the above-grade structure to resist wind or seismic forces;
- to provide a moisture-resistant barrier to below-ground habitable space in accordance with the building code; and
- to isolate nonmoisture-resistant building materials from the ground.

In some cases, masonry or concrete foundation walls incorporate a nominal amount of steel reinforcement to control cracking. Engineering specifications generally require reinforcement of concrete or masonry foundation walls because of somewhat arbitrary limits on minimum steel-to-concrete ratios, even for "plain" concrete walls. However, residential foundation walls are generally constructed of unreinforced or nominally reinforced concrete or masonry or of preservative-treated wood. The nominal reinforcement approach has provided many serviceable structures. This section discusses the issue of reinforcement and presents rational design approach for residential concrete and masonry foundation walls.

In most cases, a design for concrete or concrete masonry walls can be selected from the prescriptive tables in the applicable residential building code or the *International One- and Two-Family Dwelling Code* (ICC, 1998). Sometimes, a specific design applied with reasonable engineering judgment results in a more efficient and economical solution than that prescribed by the codes. The designer may elect to design the wall as either a reinforced or plain concrete wall. The following sections detail design methods for both wall types.

4.5.1 Concrete Foundation Walls

Regardless of the type of concrete foundation wall selected, the designer needs to determine the nominal and factored loads that in turn govern the type of wall (i.e., reinforced or unreinforced) that may be appropriate for a given application. Based on Table 3.1 of Chapter 3, the following LRFD load combinations are suggested for the design of residential concrete foundation walls:

- 1.2 D + 1.6 H
- 1.2 D + 1.6 H + 1.6 L + 0.5 (L_r or S)
- 1.2 D + 1.6 H + 1.6 (L_r or S) + 0.5 L

In light-frame homes, the first load combination typically governs foundation wall design. Axial load increases moment capacity of concrete walls when they are not appreciably eccentric, as is the case in typical residential construction.

To simplify the calculations further, the designer may conservatively assume that the foundation wall acts as a simple span beam with pinned ends, although such an assumption will tend to overpredict the stresses in the wall. In any event, the simple span model requires the wall to be adequately supported at its top by the connection to the floor framing and at its base by the connection to the footing or bearing against a basement floor slab. Appendix A contains basic load diagrams and beam equations to assist the designer in analyzing typical loading conditions and element-based structural actions encountered in residential design. Once the loads are known, the designer can perform design checks for various stresses by following ACI-318 and the recommendations contained herein.

As a practical consideration, residential designers need to keep in mind that concrete foundation walls are typically 6, 8, or 10 inches thick (nominal). The typical concrete compressive strength used in residential construction is 2,500 or 3,000 psi, although other strengths are available. Typical reinforcement tensile yield strength is 60,000 psi (Grade 60) and is primarily a matter of market supply. Refer to Section 4.2.1 for more information on concrete and steel reinforcement material properties.

4.5.1.1 Plain Concrete Wall Design

ACI-318 allows the design of plain concrete walls with some limits as discussed in ACI-318•22.0. ACI-318 recommends the incorporation of contraction and isolation joints to control cracking; however, this is not a typical practice for residential foundation walls and temperature and shrinkage cracking is practically unavoidable. It is considered to have a negligible impact on the structural integrity of a residential wall. However, cracking may be controlled (i.e., minimize potential crack widening) by reasonable use of horizontal reinforcement.

ACI-318 limits plain concrete wall thickness to a minimum of 7.5 inches; however, the *International One- and Two-Family Dwelling Code* (ICC, 1998)

permits nominal 6-inch-thick foundation walls when the height of unbalanced fill is less than a prescribed maximum. The 7.5-inch-minimum thickness requirement is obviously impractical for a short concrete stem wall as in a crawl space foundation.

Adequate strength needs to be provided and should be demonstrated by analysis in accordance with the ACI-318 design equations and the recommendations of this section. Depending on soil loads, analysis should confirm conventional residential foundation wall practice in typical conditions. Refer to Example 4.3 of Section 4.9 for an illustration of a plain concrete foundation wall design.

The following checks are used to determine if a plain concrete wall has adequate strength.

Shear Capacity

Shear stress is a result of the lateral loads on a structure associated with wind, earthquake, or backfill forces. Lateral loads are, however, either normal to the wall surface (i.e., perpendicular or out of plane) or parallel to the wall surface (i.e., in plane). The designer must consider both perpendicular and parallel shear in the wall.

Perpendicular shear is rarely a controlling factor in the design of residential concrete foundation walls. Parallel shear is also usually not a controlling factor in residential foundation walls.

If greater shear capacity is required in a plain concrete wall, it may be obtained by increasing the wall thickness or increasing the concrete compressive strength. Alternatively, a wall can be reinforced in accordance with Section 4.5.1.2.

The following equations apply to both perpendicular and parallel shear in conjunction with Figure 4.3 for plain concrete walls. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.

[ACI-318•22.5.4]

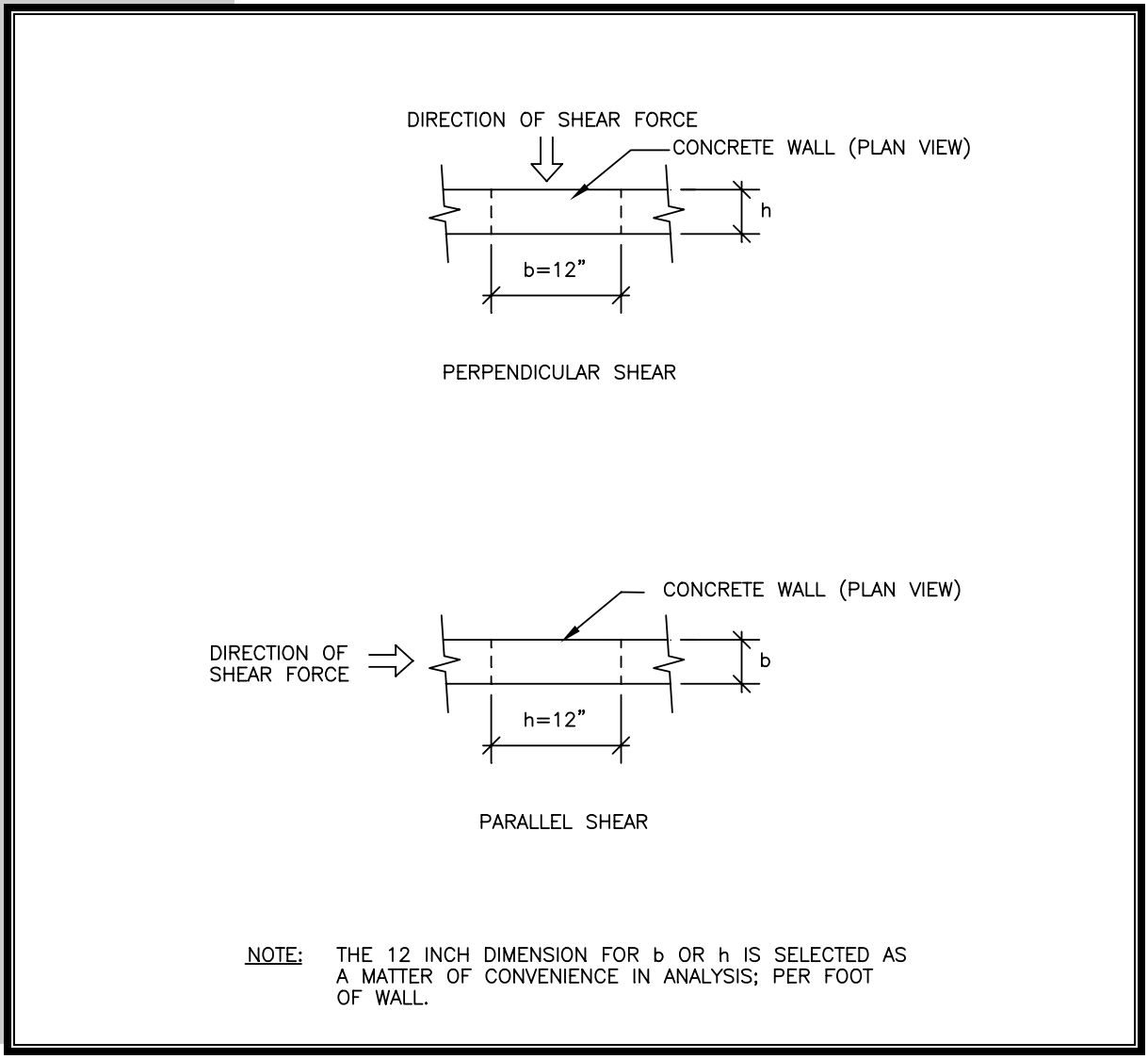
$$V_u \le \phi V_n$$

V_u = maximum factored shear load on the wall

$$\phi V_n = \phi \frac{4}{3} \sqrt{f'_c}\, bh$$

$$\phi = 0.65$$

FIGURE 4.3 *Variables Defined for Shear Calculations in Plain Concrete Walls*



Combined Axial and Bending Capacity

The ACI-318 equations listed below account for the combined effects of axial load and bending moment on a plain concrete wall. The intent is to ensure that the concrete face in compression and the concrete face in tension resulting from factored nominal axial and bending loads do not exceed the factored nominal capacity for concrete. A method of plotting the interaction equation below is shown in Example 4.4 of Section 4.9; refer to Section 4.5.1.3 for information on interaction diagrams.

[ACI-318•22.5.3, 22.6.3]

$$\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \le 1 \text{ on the compression face}$$

$$\frac{M_u}{S} - \frac{P_u}{A_g} \le 5\phi\sqrt{f_c'}$$ on the tension face

$M_u > M_{u,min}$

M_u = maximum factored nominal moment on wall

$$M_{u,min} = 0.1hP_u$$

$$M_n = 0.85f'_c S$$

$$P_n = 0.6f'_c\left[1-\left(\frac{l_c}{32h}\right)^2\right]A_g$$

P_u = factored nominal axial load on the wall at point of maximum moment

$\phi = 0.65$

Even though a plain concrete wall often calculates as adequate, the designer may elect to add a nominal amount of reinforcement for crack control or other reasons. Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through increased wall thickness or increased concrete compressive strength. Alternatively, the wall may be reinforced in accordance with Section 4.5.1.2. Walls determined to have adequate strength to withstand shear and combined axial load and bending moment may also be checked for deflection, but this is usually not a limiting factor for typical residential foundation walls.

4.5.1.2 Reinforced Concrete Design

ACI-318 allows two approaches to the design of reinforced concrete with some limits on wall thickness and the minimum amount of steel reinforcement; however, ACI-318 also permits these requirements to be waived in the event that structural analysis demonstrates adequate strength and stability in accordance with ACI-318•14.2.7. Refer to Examples 4.5, 4.6, and 4.7 in Section 4.9 for the design of a reinforced concrete foundation wall.

Reinforced concrete walls should be designed in accordance with ACI-318•14.4 by using the strength design method. The following checks for shear and combined flexure and axial load determine if a wall is adequate to resist the applied loads.

Shear Capacity

Shear stress is a result of the lateral loads on a structure associated with wind, earthquake, or lateral soil forces. The loads are, however, either normal to the wall surface (i.e., perpendicular or out of plane) or parallel to the wall surface (i.e., in plane). The designer must check both perpendicular and parallel shear in the wall to determine if the wall can resist the lateral loads present.

Perpendicular shear is rarely a controlling factor in the design of typical residential foundation concrete walls. The level of parallel shear is also usually not a controlling factor in residential foundation walls.

If greater shear capacity is required, it may be obtained by increasing the wall thickness, increasing the concrete compressive strength, adding horizontal

shear reinforcement, or installing vertical reinforcement to resist shear through shear friction. Shear friction is the transfer of shear through friction between two faces of a crack. Shear friction also relies on resistance from protruding portions of concrete on either side of the crack and by dowel action of the reinforcement that crosses the crack. The maximum limit on reinforcement spacing of 12 or 24 inches specified in ACI-318•11.5.4 is considered to be an arbitrary limit. When reinforcement is required, 48 inches as an adequate maximum spacing for residential foundation wall design agrees with practical experience.

The following equations provide checks for both perpendicular and parallel shear in conjunction with Figure 4.4. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.

[ACI-318•11.5,11.7, 11.10]

$$V_u \le \phi V_n$$

V_u = maximum factored shear load on wall

$$V_n = V_c + V_s$$

$$V_c = 2\sqrt{f'_c}\, b_w d$$

$$V_s = \frac{A_v f_y d}{s} \le 8\sqrt{f'_c}\, b_w d \quad \text{when} \quad V_u > \phi V_c$$

$$\phi = 0.85$$

Shear-Friction Method

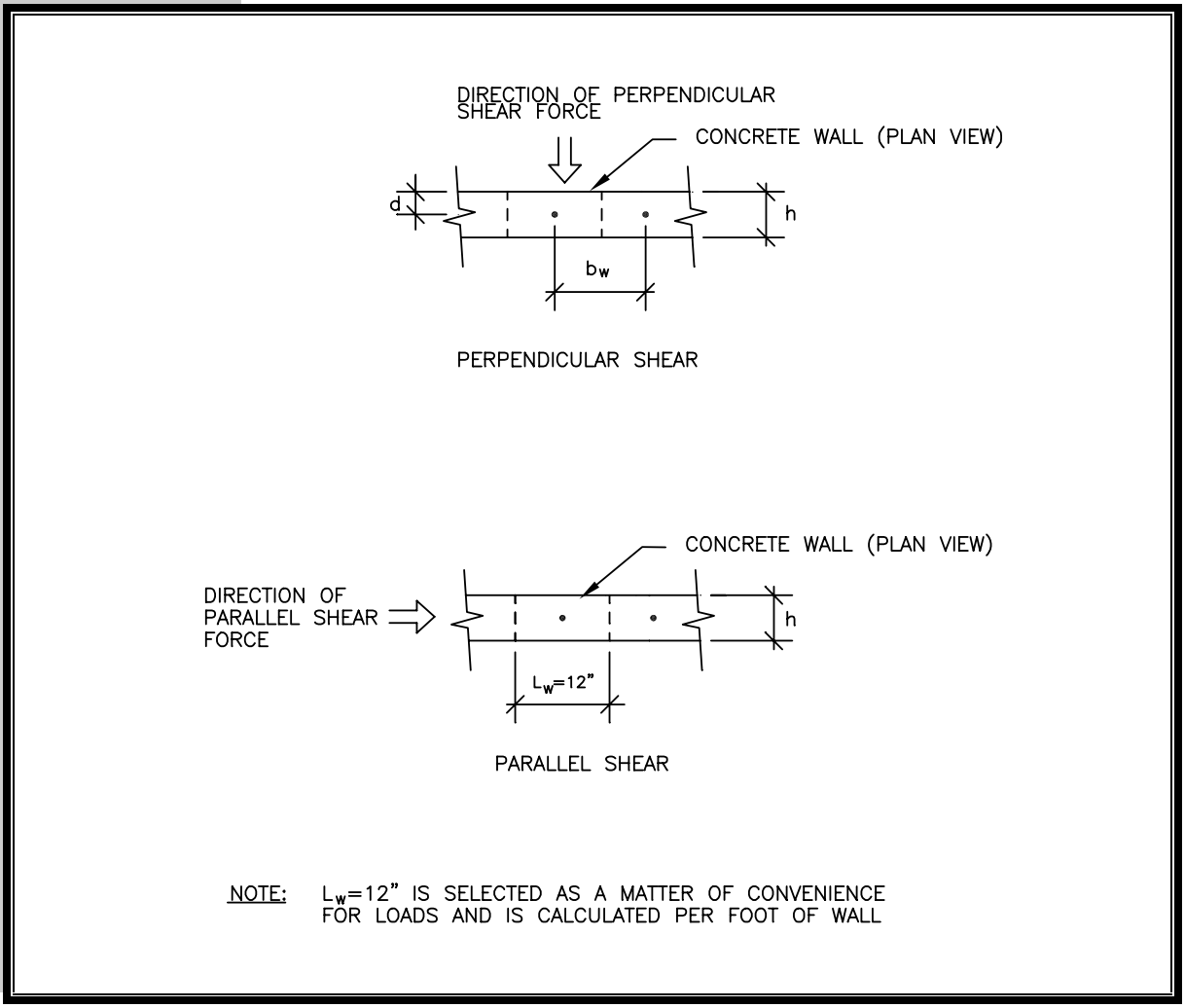
$$V_u \le \phi V_n$$

$$V_n = A_{vf} f_y \mu \le 0.2 f'_c A_c \quad \text{and} \quad \le 800 A_c$$

$$A_c = b_w h$$

$$\phi = 0.85$$

FIGURE 4.4 Variables Defined for Shear Calculations in Reinforced Concrete Walls



Combined Flexural and Axial Load Capacity

ACI-318 prescribes reinforcement requirements for concrete walls. Foundation walls commonly resist both an applied axial load from the structure above and an applied lateral soil load from backfill. To ensure that the wall's strength is sufficient, the designer must first determine slenderness effects (i.e., Euler buckling) in the wall. ACI-318•10.10 provides an approximation method to account for slenderness effects in the wall; however, the slenderness ratio must not be greater than 100. The slenderness ratio is defined in the following section as the ratio between unsupported length and the radius of gyration. In residential construction, the approximation method, more commonly known as the moment magnifier method, is usually adequate because slenderness ratios are typically less than 100 in foundation walls.

The moment magnifier method is based on the wall's classification as a "sway frame" or "nonsway frame." In concept, a sway frame is a frame (i.e., columns and beams) as opposed to a concrete bearing wall system. Sway frames are not discussed in detail herein because the soil pressures surrounding a

residential foundation typically provide lateral support to resist any racking and deflections associated with a sway frame. More important, foundation walls generally have few openings and thus do not constitute a framelike system. For more information on sway frames and their design procedure, refer to ACI-318•10.13.

The moment magnifier method uses the relationship of the axial load and lateral load in addition to wall thickness and unbraced height to determine a multiplier of 1 or greater, which accounts for slenderness in the wall. The multiplier is termed the moment magnifier. It magnifies the calculated moment in the wall resulting from the lateral soil load and any eccentricity in axial load. Together, the axial load and magnified moment are used to determine whether the foundation wall section is adequate to resist the applied loads. The following steps are required to determine the amount of reinforcement required in a typical residential concrete foundation wall to resist combined flexure and axial loads:

- calculate axial and lateral loads;
- verify that the nonsway condition applies;
- calculate slenderness;
- calculate the moment magnifier; and
- plot the axial load and magnified moment on an interaction diagram.

The following sections discuss the procedure in detail.

Slenderness

Conservatively, assuming that the wall is pinned at the top and bottom, slenderness in the wall can be calculated by using the equation below. The effective length factor *k* is conservatively assumed to equal 1 in this condition. It should be noted that a value of *k* much less than 1 (i.e., 0.7) may actually better represent the end conditions (i.e., nonpinned) of residential foundation walls.

[ACI-318•10.10]

$$\frac{kl_u}{r} < 34 \qquad \text{slenderness ratio}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{bd^3/12}{bd}} = \sqrt{\frac{d^2}{12}} \qquad \text{radius of gyration}$$

Moment Magnifier Method

The moment magnifier method is an approximation method allowed in ACI-318•10.10 for concrete walls with a slenderness ratio less than or equal to 100. If the slenderness ratio is less than 34, then the moment magnifier is equal to 1 and requires no additional analysis. The design procedure and equations below follow ACI-318•10.12. The equation for *EI*, as listed in ACI-318, is applicable to walls containing a double layer of steel reinforcement. Residential walls typically contain only one layer of steel reinforcement; therefore, the equation for *EI*, as listed herein, is based on Section 10.12 (ACI, 1996).

[ACI-318•10.12.3]

$$M_{u,mag} = \delta M_u \quad \Leftarrow \text{ Magnified Moment}$$

$$\delta = \frac{C_m}{1 - \left(P_u \Big/ 0.75P_c \right)} \geq 1$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

$$C_m = 0.6$$

or

$$C_m = 1 \text{ for members with transverse loads between supports}$$

$$M_{u,min} = P_u (0.6 + 0.03h)$$

$$EI = \frac{0.4E_c I_g}{\beta} \geq \frac{E_c I_g \left(0.5 - e/h\right)}{\beta} \geq \frac{0.1E_c I_g}{\beta}$$

$$e = \frac{M_2}{P_u}$$

$$\beta = 0.9 + 0.5\beta_d^2 - 12\rho \geq 1.0$$

$$\rho = \frac{A_s}{A_g}$$

$$\beta_d = \frac{P_{u,dead}}{P_u}$$

$$E_c = 57{,}000\sqrt{f'_c} \text{ or } w_c^{1.5} 33\sqrt{f'_c}$$

Given that the total factored axial load in residential construction typically falls below 3,000 pounds per linear foot of wall and that concrete compressive strength is typically 3,000 psi, Table 4.4 provides prescriptive moment magnifiers. Interpolation is permitted between wall heights and between factored axial loads. Depending on the reinforcement ratio and the eccentricity present, some economy is lost in using the Table 4.4 values instead of the above calculation method.

TABLE 4.4 Simplified Moment Magnification Factors, δ_{ns}

Minimum Wall Thickness (inches)	Maximum Wall Height (feet)	Factored Axial Load (plf)	
		2,000	4,000
5.5	**8**	1.07	1.15
	10	1.12	1.26
7.5	**8**	1.03	1.06
	10	1.04	1.09
9.5	**8**	1.00	1.03
	10	1.00	1.04

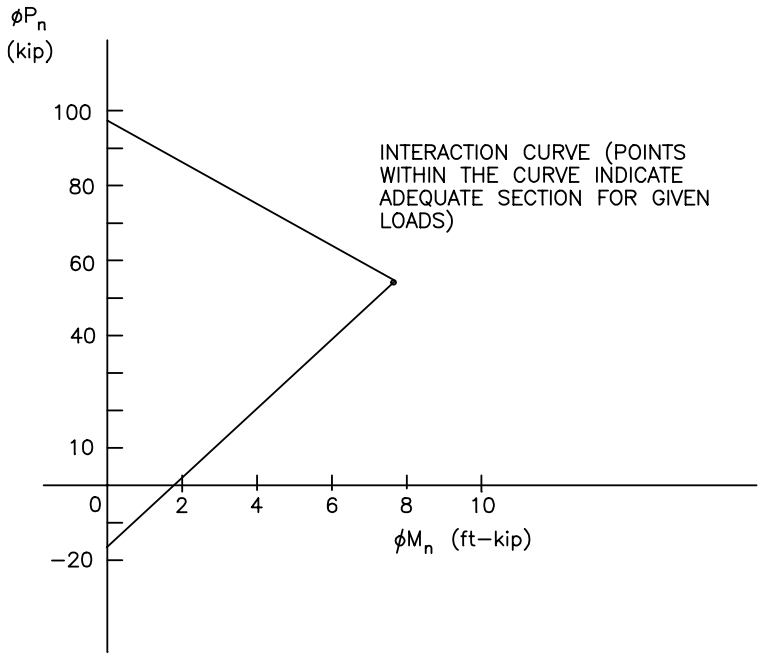
Example 4.6 in Section 4.9 presents the complete design of a reinforced concrete foundation wall. The magnified moment and corresponding total factored axial load are plotted on an interaction diagram as shown in Example 4.7. Refer to Section 4.5.1.3 for a description of interaction diagrams and additional resources.

4.5.1.3 Interaction Diagrams

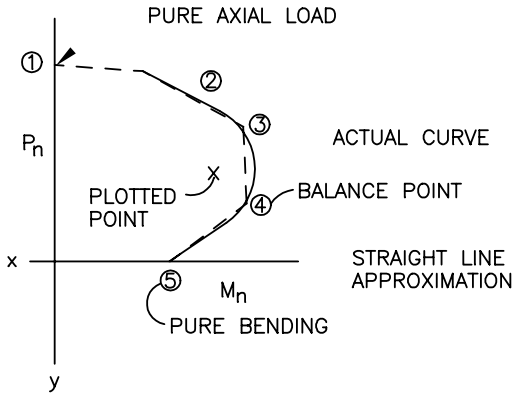
An interaction diagram is a graphic representation of the relationship between the axial load and bending capacity of a reinforced or plain concrete wall. The primary use of interaction diagrams is as a design aid for selecting predetermined concrete wall or column designs for varying loading conditions. Several publications provide interaction diagrams for use with concrete. These publications, however, typically focus on column or wall design that is heavily reinforced in accordance with design loads common in commercial construction. Residential concrete walls are either plain or slightly reinforced with one layer of reinforcement typically placed near the center of the wall. Plain and reinforced concrete interaction diagrams for residential applications and the methods for deriving them may be found in *Structural Design of Insulating Concrete Form Walls in Residential Construction* (PCA, 1998). PCA also offers a computer program that plots interaction diagrams based on user input; the program is entitled *PCA Column* (PCACOL).

An interaction diagram assists the designer in determining the wall's structural adequacy at various loading conditions (i.e., combinations of axial and bending loads). Figure 4.5 illustrates interaction diagrams for plain and reinforced concrete. Both the design points located within the interaction curve for a given wall height and the reference axes represent a combination of axial load and bending moment that the wall can safely support. The most efficient design is close to the interaction diagram curve. For residential applications, the designer, realizing that the overall design process is not exact, usually accepts designs within plus or minus 5 percent of the interaction curve.

FIGURE 4.5 *Typical Interaction Diagrams for Plain and Reinforced Concrete Walls*



PLAIN CONCRETE WALL



REINFORCED CONCRETE WALL

4.5.1.4 Minimum Concrete Wall Reinforcement

Plain concrete foundation walls provide serviceable structures when they are adequately designed (see Section 4.5.1.1). However, when reinforcement is used to provide additional strength in thinner walls or to address more heavily loaded conditions, tests have shown that horizontal and vertical wall reinforcement spacing limited to a maximum of 48 inches on center results in performance that agrees reasonably well with design expectations (Roller, 1996).

ACI-318•22.6.6.5 requires two No. 5 bars around all wall openings. As an alternative more suitable to residential construction, a minimum of one rebar should be placed on each side of openings between 2 and 4 feet wide and two rebars on each side and one on the bottom of openings greater than 4 feet wide. The rebar should be the same size required by the design of the reinforced wall or a minimum No. 4 for plain concrete walls. In addition, a lintel (i.e., concrete beam) is required at the top of wall openings; refer to Section 4.5.1.6 for more detail on lintels.

4.5.1.5 Concrete Wall Deflection

ACI-318 does not specifically limit wall deflection. Therefore, deflection is usually not analyzed in residential foundation wall design. Regardless, a deflection limit of $L/240$ for unfactored soil loads is not unreasonable for below-grade walls.

When using the moment magnifier method, the designer is advised to apply the calculated moment magnification factor to the unfactored load moments used in conducting the deflection calculations. The calculation of wall deflection should also use effective section properties based on $E_c I_g$ for plain concrete walls and $E_c I_e$ for reinforced concrete walls; refer to ACI 318•9.5.2.3 to calculate the effective moment of inertia, I_e.

If unfactored load deflections prove unacceptable, the designer may increase the wall thickness or the amount of vertical wall reinforcement. For most residential loading conditions, however, satisfying reasonable deflection requirements should not be a limiting condition.

4.5.1.6 Concrete Wall Lintels

Openings in concrete walls are constructed with concrete, steel, precast concrete, cast stone, or reinforced masonry wall lintels. Wood headers are also used when not supporting concrete construction above and when continuity at the top of the wall (i.e., bond beam) is not critical, as in high-hazard seismic or hurricane coastal zones, or is maintained sufficiently by a wood sill plate and other construction above.

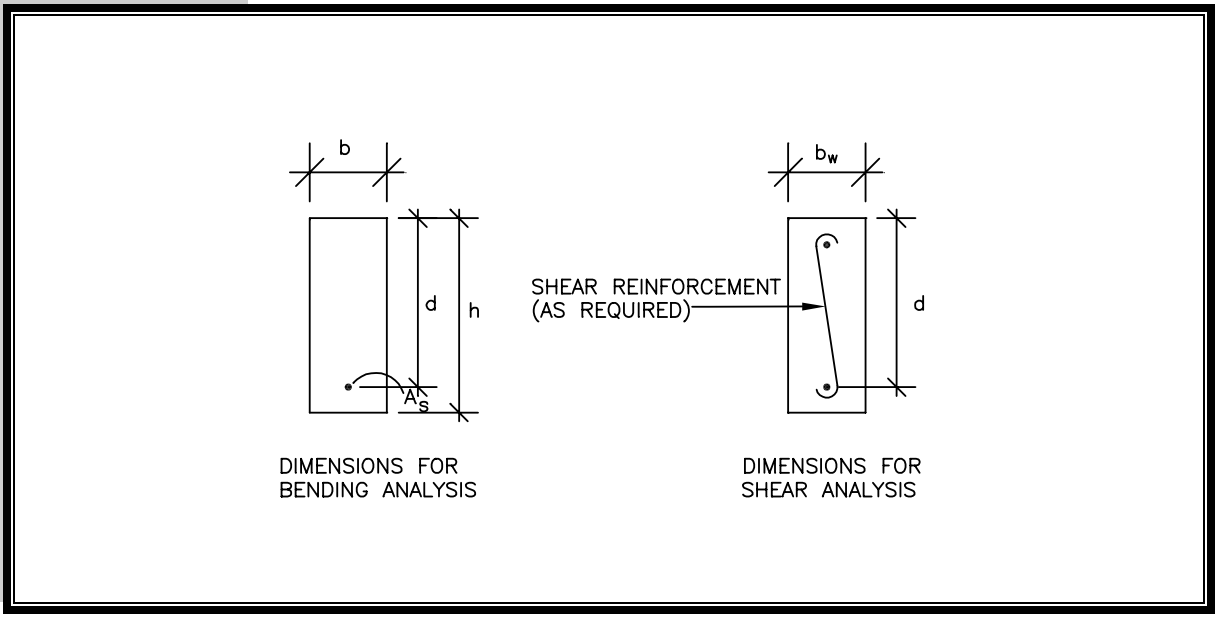
This section focuses on the design of concrete lintels in accordance with Chapters 10 and 11 of ACI-318. The concrete lintel is often assumed to act as a simple span with each end pinned. However, the assumption implies no top reinforcement to transfer the moment developed at the end of the lintel. Under that condition, the lintel is assumed to be cracked at the ends such that the end

moment is zero and the shear must be transferred from the lintel to the wall through the bottom reinforcement.

If the lintel is assumed to act as a fixed-end beam, sufficient embedment of the top and bottom reinforcement beyond each side of the opening should be provided to fully develop a moment-resisting end in the lintel. Though more complicated to design and construct, a fixed-end beam reduces the maximum bending moment (i.e., $wl^2/12$ instead of $wl^2/8$) on the lintel and allows increased spans. A concrete lintel cast in a concrete wall acts somewhere between a true simple span beam and a fixed-end beam. Thus, a designer may design the bottom bar for a simple span condition and the top bar reinforcement for a fixed-end condition (conservative). Often, a No. 4 bar is placed at the top of each wall story to help tie the walls together (bond beam) which can also serve as the top reinforcement for concrete lintels. Figure 4.6 depicts the cross section and dimensions for analysis of concrete lintels. Example 4.8 demonstrates the design of a concrete lintel; refer to Section 4.9.

For additional information on concrete lintels and their design procedure, refer to the *Structural Design of Insulating Concrete Form Walls in Residential Construction* (PCA, 1998) and to *Testing and Design of Lintels Using Insulating Concrete Forms* (HUD, 2000). The latter, demonstrates through testing that shear reinforcement (i.e., stirrups) of concrete lintels is not necessary for short spans (i.e., 3 feet or less) with lintel depths of 8 inches or more. This research also indicates that the minimum reinforcement requirements in ACI-318 for beam design are conservative when a minimum #4 rebar is used as bottom reinforcement. Further, lintels with small span-to-depth ratios can be accurately designed as deep beams in accordance with ACI-318 when the minimum reinforcement ratios are met; refer to ACI-318•11.4.

FIGURE 4.6 Design Variables Defined for Lintel Bending and Shear



Flexural Capacity

The following equations are used to determine the flexural capacity of a reinforced concrete lintel in conjunction with Figure 4.6. An increase in the lintel depth or area of reinforcement is suggested if greater bending capacity is required. As a practical matter, though, lintel thickness is limited to the thickness of the wall in which a lintel is placed. In addition, lintel depth is often limited by the floor-to-floor height and the vertical placement of the opening in the wall. Therefore, in many cases, increasing the amount or size of reinforcement is the most practical and economical solution.

[ACI-318•10]

$$M_u \le \phi M_n$$

$$M_u = \frac{w\ell^2}{12} \text{ for fixed-end beam model}$$

$$M_u = \frac{w\ell^2}{8} \text{ for simple span beam model}$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$\phi = 0.9$$

Shear Capacity

Concrete lintels are designed for shear resulting from wall, roof, and floor loads in accordance with the equations below and Figure 4.6.

[ACI-318•11]

$$V_u \le \phi V_n$$

$$V_n = V_c + V_s$$

$$V_c = 2\sqrt{f'_c}\, b_w d$$

$$V_s = \frac{A_v f_y d}{s} \le 8\sqrt{f'_c} b_w d \text{ when } V_u > \phi V_c$$

$$A_{v,min} = \frac{50 b_w s}{f_y} \text{ when } V_u > \frac{\phi V_c}{2}$$

$$s \le \text{minimum of } \left\{ d/2 \text{ or } 24 \text{ in} \right\}$$

$$s \le \text{minimum of } \left\{ d/4 \text{ or } 12 \text{ in} \right\} \text{ when } V_s > 4\sqrt{f'_c} b_w d$$

$$\phi = 0.85$$

Check Concrete Lintel Deflection

ACI-318 does not specifically limit lintel deflection. Therefore, a reasonable deflection limit of *L/240* for unfactored live loads is suggested. The selection of an appropriate deflection limit, however, is subject to designer

discretion. In some applications, a lintel deflection limit of *L/180* with live and dead loads is adequate. A primary consideration is whether lintel is able to move independently of door and window frames. Calculation of lintel deflection should use unfactored loads and the effective section properties E_cI_e of the assumed concrete section; refer to ACI-318•9.5.2.3 to calculate the effective moment of inertia I_e of the section.

4.5.2 Masonry Foundation Walls

Masonry foundation wall construction is common in residential construction. It is used in a variety of foundation types, including basements, crawl spaces, and slabs on grade. For prescriptive design of masonry foundation walls in typical residential applications, a designer or builder may use the *International One- and Two-Family Dwelling Code* (ICC, 1998) or the local residential building code.

ACI-530 provides for the design of masonry foundation walls by using allowable stress design (ASD). Therefore, design loads may be determined according to load combinations presented in Chapter 3 as follows:

- D + H
- D + H + L + 0.3 (L_r or S)
- D + H + (L_r or S) + 0.3 L

In light-frame homes, the first load combination typically governs masonry walls for the same reasons stated in Section 4.5.1 for concrete foundation walls. To simplify the calculations, the designer may conservatively assume that the wall story acts as a simple span with pinned ends, although such an assumption may tend to overpredict the stresses in the wall. For a discussion on calculating the loads on a structure, refer to Chapter 3. Appendix A contains basic load diagrams and equations to assist the designer in calculating typical loading conditions and element-based structural actions encountered in residential design. Further, walls that are determined to have adequate strength to withstand shear and combined axial load and bending moment generally satisfy unspecified deflection requirements. Therefore, foundation wall deflection is not discussed in this section. However, if desired, deflection may be considered as discussed in Section 4.5.1.5 for concrete foundation walls.

To follow the design procedure, the designer needs to know the strength properties of various types and grades of masonry, mortar, and grout currently available on the market; Section 4.2.2 discusses the material properties. With the loads and material properties known, the designer can then perform design checks for various stresses by following ACI-530. Residential construction rarely involves detailed masonry specifications but rather makes use of standard materials and methods familiar to local suppliers and trades.

An engineer's inspection of a home is hardly ever required under typical residential construction conditions. Designers should be aware, however, that in jurisdictions covered by the *Uniform Building Code* (ICBO, 1997), lack of inspection on the jobsite requires reductions in the allowable stresses to account for potentially greater variability in material properties and workmanship. Indeed,

a higher level of inspection should be considered when masonry construction is specified in high-hazard seismic or severe hurricane areas. ACI-530 makes no distinction between inspected and noninspected masonry walls and therefore does not require adjustments in allowable stresses based on level of inspection.

As a residential designer, keep in mind that concrete masonry units (i.e., block) are readily available in nominal 6-, 8-, 10-, and 12-inch thicknesses. It is generally more economical if the masonry unit compressive strength f'_m ranges between 1,500 and 3,000 psi. The standard block used in residential and light commercial construction is usually rated at 1,900 psi.

4.5.2.1 Unreinforced Masonry Design

ACI-530 addresses the design of unreinforced masonry to ensure that unit stresses and flexural stresses in the wall do not exceed certain maximum allowable stresses. It provides for two methods of design: an empirical design approach and an allowable stress design approach.

Walls may be designed in accordance with ACI-530•5 by using the empirical design method under the following conditions:

- The building is not located in Seismic Design Category D or E as defined in NEHRP-97 or ASCE 7-98 (i.e., Seismic Zones 3 or 4 in most current and local building codes); refer to Chapter 3.
- Foundation walls do not exceed 8 feet in unsupported height.
- The length of the foundation walls between perpendicular masonry walls or pilasters is a maximum of 3 times the basement wall height. This limit typically does not apply to residential basements as required in the *International One- and Two-Family Dwelling Code* (ICC, 1998) and other similar residential building codes.
- Compressive stresses do not exceed the allowable stresses listed in ACI-530; compressive stresses are determined by dividing the design load by the gross cross-sectional area of the unit per ACI-530•5.4.2.
- Backfill heights do not exceed those listed in Table 4.5.
- Backfill material is nonexpansive and is tamped no more than necessary to prevent excessive settlement.
- Masonry is laid in running bond with Type M or S mortar.
- Lateral support is provided at the top of the foundation wall before backfilling.

Drainage is important when using the empirical table because lack of good drainage may substantially increase the lateral load on the foundation wall if the soil becomes saturated. As required in standard practice, the finish grade around the structure should be adequately sloped to drain surface water away from the foundation walls. The backfill material should also be drained to remove ground water from poorly drained soils.

Wood floor framing typically provides lateral support to the top of masonry foundation walls and therefore should be adequately connected to the masonry in accordance with one of several options. The most common method of

connection calls for a wood sill plate, anchor bolts, and nailing of the floor framing to the sill plate (see Chapter 7).

When the limits of the empirical design method are exceeded, the allowable stress design procedure for unreinforced masonry, as detailed below, provides a more flexible approach by which walls are designed as compression and bending members in accordance with ACI-530•2.2.

TABLE 4.5 Nominal Wall Thickness for 8-Foot-High Masonry Foundation Walls[1,2]

Nominal Wall Thickness	Maximum Unbalanced Backfill Height		
	Hollow Unit Masonry	Solid Unit Masonry	Fully Grouted Unit Masonry
6 inches	3	5	5
8 inches	5	5	7
10 inches	6	7	8
12 inches	7	7	8

Source: Modified from the ACI-530• 9.6 by using the International One-and Two-Family Dwelling Code (ICC, 1998).

Notes:

[1]Based on a backfill with an assumed equivalent fluid density of 30 pcf.

[2]Backfill height is measured from the top of the basement slab to the finished exterior grade; wall height is measured from the top of the basement slab to the top of the wall.

Walls may be designed in accordance with ACI-530•2.2 by using the allowable stress design method. The fundamental assumptions, derivation of formulas, and design procedures are similar to those developed for strength-based design for concrete except that the material properties of masonry are substituted for those of concrete. Allowable masonry stresses used in allowable stress design are expressed in terms of a fraction of the specified compressive strength of the masonry at the age of 28 days, f'_m. A typical fraction of the specified compressive strength is 0.25 or 0.33, which equates to a conservative safety factor between 3 and 4 relative to the minimum specified masonry compressive strength. Design values for flexural tension stress are given in Table 4.6. The following design checks are used to determine if an unreinforced masonry wall is structurally adequate (refer to Example 4.9 for the design of an unreinforced concrete masonry wall).

TABLE 4.6 *Allowable Flexural Tension Stresses F_a for Allowable Stress Design of Unreinforced Masonry*

Type of Masonry Unit Construction	Mortar Type M or S	
	Portland Cement/Lime (psi)	Masonry Cement and Air-Entrained Portland Cement/Lime (psi)
Normal to Bed Joints		
Solid	40	24
Hollow[1]		
Ungrouted	25	15
Fully grouted	68	41
Parallel to Bed Joints in Running Bond		
Solid	80	48
Hollow		
Ungrouted/partially grouted	50	30
Fully grouted	80	48

Source: Table 6.3.1.1 in ACI-530•6.0.

Note:

[1]For partially grouted masonry, allowable stresses may be determined on the basis of linear interpolation between fully grouted and ungrouted hollow units based on the amount of grouting.

Shear Capacity

Shear stress is a result of the lateral loads on the structure associated with wind, earthquakes, or backfill forces. Lateral loads are both normal to the wall surface (i.e., perpendicular or out of plane) and parallel to the wall surface (i.e., parallel or in plane). Both perpendicular and parallel shear should be checked; however, neither perpendicular nor parallel shear is usually a controlling factor in residential foundation walls.

If greater perpendicular shear capacity is required, it may be obtained by increasing the wall thickness, increasing the masonry unit compressive strength, or adding vertical reinforcement in grouted cells. If greater parallel shear capacity is required, it may be obtained by increasing the wall thickness, reducing the size or number of wall openings, or adding horizontal joint reinforcement. Horizontal truss-type joint reinforcement can substantially increase parallel shear capacity, provided that it is installed properly in the horizontal mortar bed joints. If not installed properly, it can create a place of weakness in the wall, particularly in out-of-plane bending of an unreinforced masonry wall.

The equations below are used to check perpendicular and parallel shear in masonry walls. The variable N_v is the axial design load acting on the wall at the point of maximum shear. The equations are based on A_n, which is the net cross-sectional area of the masonry. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.

[ACI-530•2.2.5]

$$f_v \le F_v$$

$$f_v = \frac{3V}{2A_n}$$

$$F_v = \text{minimum of} \begin{cases} 1.5\sqrt{f'_m} & \text{for axial and shear members} \\ 120\text{psi} & \\ 37\text{ psi} + 0.45\,\frac{N_v}{A_n} & \text{for running bond} \end{cases}$$

Axial Compression Capacity

The following equations from ACI-530•2.3 are used to design masonry walls and columns for compressive loads only. They are based on the net cross-sectional area of the masonry, including grouted and mortared areas.

[ACI-530•2.3]

Columns

$$P \le P_a$$

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{where } h/r \le 99$$

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } h/r > 99$$

$$P_{a,maximum} = F_a A_n$$

$$r = \sqrt{\frac{I}{A_n}}$$

Walls

$$f_a \le F_a$$

$$F_a = \left(0.25f'_m\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \text{ where } h/r \le 99$$

$$f_a = P/A$$

$$F_a = \left(0.25f'_m\right)\left(\frac{70r}{h}\right)^2 \text{ where } h/r > 99$$

$$r = \sqrt{\frac{I}{A_n}} \cong \frac{t}{\sqrt{12}}$$

$$P_e = \frac{TT^2 E_m I}{h^2}\ (1-0.577\frac{e}{r})^3$$

$P < 1/4\ P_e$

$E_m = 900\ F'_m$

Combined Axial Compression and Flexural Capacity

The following equations from ACI-530 determine the relationship of the combined effects of axial load and bending moment on a masonry wall.

[ACI-530•2.3]

$$\frac{f_a}{F_a}+\frac{f_b}{F_b}\le 1$$

$$f_a=\frac{P}{A_n}$$

$$P\le 0.25P_e$$

$$F_a=\left(0.25f'_m\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \text{ for } h/r \le 99$$

$$F_a=\left(0.25f'_m\right)\left(\frac{70r}{h}\right)^2 \text{ for } h/r > 99$$

$$r=\sqrt{\frac{I}{A_n}}$$

$$f_b=\frac{M}{S}$$

$$F_b=0.33f'_m$$

$$P_e=\frac{\pi^2 E_m I}{h^2}\left(1-0.577\frac{e}{r}\right)^3$$

$$E_m=900f'_m$$

$f_t < F_t$

F_t = ACI-530 Table 2.2.3.2

$$f_t = \frac{-P}{A_n}+\frac{M}{S}$$

Tension Capacity

ACI-530 provides allowable values for flexural tension transverse to the plane of a masonry wall. Standard principles of engineering mechanics determine the tension stress due to the bending moment caused by lateral (i.e., soil) loads and offset by axial loads (i.e., dead loads).

[ACI-530•2.3]

$f_t < F_t$

F_t = ACI-530 Table 2.2.3.2

$$f_t = \frac{P}{A_n}+\frac{M}{S}$$

Even though an unreinforced masonry wall may calculate as adequate, the designer may consider adding a nominal amount of reinforcement to control cracking (refer to Section 4.5.2.3 for a discussion on nominal reinforcement).

Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through increased wall thickness, increased masonry compressive strength, or the addition of steel reinforcement.

Usually the most effective and economical solution for providing greater wall capacity in residential construction is to increase wall thickness, although reinforcement is also common. Section 4.5.2.2 discusses the design procedure for a reinforced masonry wall.

4.5.2.2 Reinforced Masonry Design

When unreinforced concrete masonry wall construction does not satisfy all design criteria (i.e., load, wall thickness limits, etc.), reinforced walls may be designed by following the allowable stress design procedure or the strength-based design procedure of ACI-530. The allowable stress design procedure outlined below describes an approach by which walls are designed in accordance with ACI-530•2.3. Although not discussed in detail herein, walls may also be designed by following the strength-based design method specified in ACI-530.

For walls designed in accordance with ACI-530•2.3 using the allowable stress design method, the fundamental assumptions, derivation of formulas, and design procedures are similar to those for design for concrete except that the material properties of masonry are substituted for those of concrete. Allowable masonry stresses used in allowable stress design are expressed in terms of a fraction of the specified compressive strength of the masonry at the age of 28 days, f'_m. A typical fraction of the specified compressive strength is 0.25, which equates to a conservative safety factor of 4. The following design checks determine if a reinforced masonry wall is structurally adequate (refer to Example 4.10 for the design of a reinforced concrete masonry wall).

Shear Capacity

Shear stress is a result of lateral loads on the structure associated with wind, earthquakes, or backfill forces. Lateral loads are both normal to the wall surface (i.e., perpendicular or out of plane) and parallel to the wall surface (i.e., parallel or in plane). Both perpendicular and parallel shear should be checked, however, perpendicular shear is rarely a controlling factor in the design of masonry walls and parallel shear is not usually a controlling factor unless the foundation is partially or fully above grade (i.e., walk-out basement) with a large number of openings.

The equations below check perpendicular and parallel shear in conjunction with Figure 4.7. Some building codes include a “j” coefficient in these equations. The “j” coefficient defines the distance between the center of the compression area and the center of the tensile steel area; however, it is often dismissed or approximated as 0.9. If greater parallel shear capacity is required, it may be obtained in a manner similar to that recommended in the previous section for unreinforced masonry design. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.

[ACI-530•7.5]

$$f_v \le F_v$$

$$f_v = \frac{V}{bd}$$

$$F_v = 1.0\sqrt{f'_m} \le 50 \text{psi} \quad \text{for flexural members}$$

$$F_v = \frac{1}{3}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \le \left(80 - 45\frac{M}{Vd}\right) \text{psi for shear walls where} \frac{M}{Vd} < 1$$

$$F_v = 1.0\sqrt{f'_m} \le 35 \text{psi for shear walls where} \frac{M}{Vd} \ge 1$$

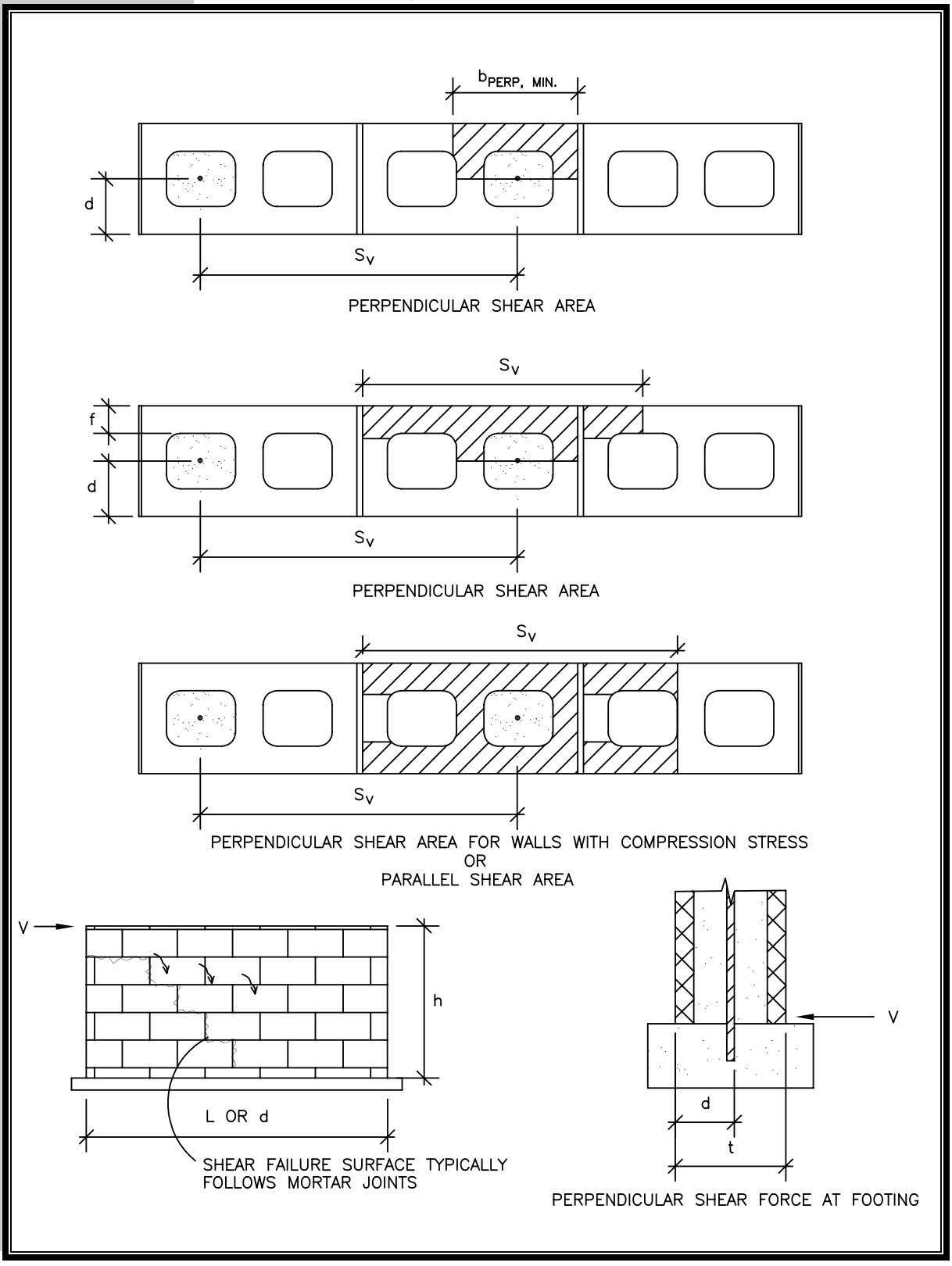
If the shear stress exceeds the above allowables for masonry only, the designer must design shear reinforcing with the shear stress equation changes in accordance with ACI-530•2.3.5. In residential construction, it is generally more economical to increase the wall thickness or to grout additional cores instead of using shear reinforcement. If shear reinforcement is desired, refer to ACI-530. ACI-530 limits vertical reinforcement to a maximum spacing *s* of 48 inches; however, a maximum of 96 inches on-center is suggested as adequate. Masonry homes built with reinforcement at 96 inches on-center have performed well in hurricane-prone areas such as southern Florida.

Flexural or axial stresses must be accounted for to ensure that a wall is structurally sound. Axial loads increase compressive stresses and reduce tension stresses and may be great enough to keep the masonry in an uncracked state under a simultaneous bending load.

Axial Compression Capacity

The following equations from ACI-530•2.3 are used to determine if a masonry wall can withstand conditions when compressive loads act only on walls and columns (i.e., interior load-bearing wall or floor beam support pier). As with concrete, compressive capacity is usually not an issue in supporting a typical light-frame home. An exception may occur with the bearing points of long-spanning beams. In such a case, the designer should check bearing capacity by using ACI-530•2.1.7.

FIGURE 4.7 *Variables Defined for Shear Calculations in Reinforced Concrete Masonry Walls*



[ACI-530•2.3]

Columns

$$P \le P_a$$

$$P_a = \left(0.25 f'_m A_n + 0.65 A_{st} F_s\right)\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{where } {}^{h}/_{r} \le 99$$

$$P_a = \left(0.25 f'_m A_n + 0.65 A_{st} F_s\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } {}^{h}/_{r} > 99$$

$$P_{a,maximum} = F_a A_n$$

$$r = \sqrt{\frac{I}{A_e}}$$

Walls

$$f_a \le F_a$$

$$F_a = \left(0.25 f'_m\right)\left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ where } {}^{h}/_{r} \le 99$$

$$F_a = \left(0.25 f'_m\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } {}^{h}/_{r} > 99$$

$$r = \sqrt{\frac{I}{A_e}}$$

Calculation using the above equations is based on A_e, which is the effective cross-sectional area of the masonry, including grouted and mortared areas substituted for A_n.

Combined Axial Compression and Flexural Capacity

In accordance with ACI-530•2.3.2, the design tensile forces in the reinforcement due to flexure shall not exceed 20,000 psi for Grade 40 or 50 steel, 24,000 psi for Grade 60 steel, or 30,000 psi for wire joint reinforcement. As stated, most reinforcing steel in the U.S. market today is Grade 60. The following equations pertain to walls that are subject to combined axial and flexure stresses.

[ACI-530•7.3]

$$F_b = 0.33 f'_m$$

$$f_b = \frac{M}{S} \le \left(1 - \frac{f_a}{F_a}\right) F_b$$

Columns

$$\frac{P}{P_a} + \frac{f_b}{F_b} \le 1$$

$$P_a = \left(0.25 f'_m A_n + 0.65 A_{st} F_s\right)\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{where } {}^{h}/_{r} \le 99$$

$$P_a = \left(0.25 f'_m A_n + 0.65 A_{st} F_s\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } {}^{h}/_{r} > 99$$

Walls

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$

$f_a = \frac{P}{A_e} \le 0.33 f'_m$ due to flexure only or flexure in combination with axial load

$$F_a = (0.25 f'_m)\left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } h/r \le 99$$

$$F_a = (0.25 f'_m)\left(\frac{70r}{h}\right)^2 \text{ for } h/r > 99$$

Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through increased wall thickness, increased masonry compressive strength, or added steel reinforcement.

4.5.2.3 Minimum Masonry Wall Reinforcement

Unreinforced concrete masonry walls have proven serviceable in millions of homes. Builders and designers may, however, wish to specify a nominal amount of reinforcement even when such reinforcement is not required by analysis. For example, it is not uncommon to specify horizontal reinforcement to control shrinkage cracking and to improve the bond between intersecting walls. When used, horizontal reinforcement is typically specified as a ladder or truss-type wire reinforcement. It is commonly installed continuously in mortar joints at vertical intervals of 24 inches (i.e., every third course of block).

For reinforced concrete masonry walls, ACI-530 stipulates minimum reinforcement limits as shown below; however, the limits are somewhat arbitrary and have no tangible basis as a minimum standard of care for residential design and construction. The designer should exercise reasonable judgment based on application conditions, experience in local practice, and local building code provisions for prescriptive masonry foundation or above-grade wall design in residential applications.

[ACI-530•2.3.5]

$$A_{s,required} = \frac{M}{F_s d}$$

$$A_{v,min} = 0.0013bt$$

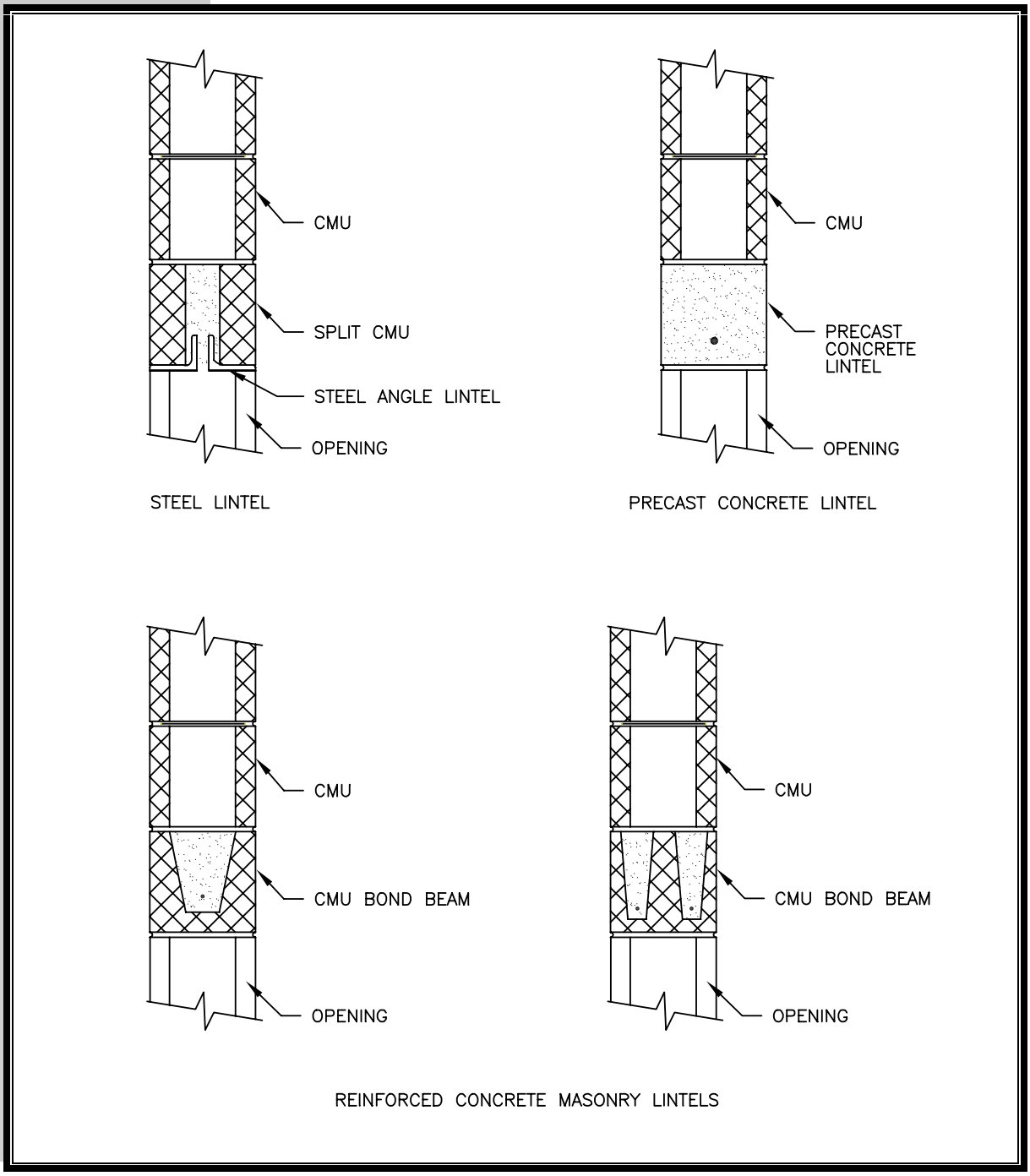
$$A_{h,min} = 0.0007bt$$

4.5.2.4 Masonry Wall Lintels

Openings in masonry walls are constructed by using steel, precast concrete, or reinforced masonry lintels. Wood headers are also used when they do not support masonry construction above and when continuity at the top of the wall (i.e., bond beam) is not required or is adequately provided within the system of wood-framed construction above. Steel angles are the simplest shapes and are

suitable for openings of moderate width typically found in residential foundation walls. The angle should have a horizontal leg of the same width as the thickness of the concrete masonry that it supports. Openings may require vertical reinforcing bars with a hooked end that is placed on each side of the opening to restrain the lintel against uplift forces in high-hazard wind or earthquake regions. Building codes typically require steel lintels exposed to the exterior to be a minimum 1/4-inch thick. Figure 4.8 illustrates some lintels commonly used in residential masonry construction.

FIGURE 4.8 Concrete Masonry Wall Lintel Types



Many prescriptive design tables are available for lintel design. For more information on lintels, arches, and their design, refer to the NCMA's TEK Notes; refer to contact information in Chapter 1. Information on lintels and arches can also be found in *Masonry Design and Detailing* (Beall, 1997).

4.5.3 Preservative-Treated Wood Foundation Walls

Preservative-treated wood foundations, commonly known as permanent wood foundations (PWF), have been used in over 300,000 homes and other structures throughout the United States. When properly installed, they provide foundation walls at an affordable cost. In some cases, the manufacturer may offer a 50-year material warranty, which exceeds the warranty offered for other common foundation materials.

A PWF is a load-bearing, preservative-treated, wood-framed foundation wall sheathed with preservative-treated plywood; it bears on a gravel spread footing. PWF lumber and plywood used in foundations is pressure treated with calcium chromium arsenate (CCA) to a minimum retention of 0.6 pcf. The walls are supported laterally at the top by the floor system and at the bottom by a cast-in-place concrete slab or pressure-treated lumber floor system or by backfill on the inside of the wall. Proper connection details are essential, along with provisions for drainage and moisture protection. All fasteners and hardware used in a PWF should be stainless steel or hot-dipped galvanized. Figure 4.9 illustrates a PWF.

PWFs may be designed in accordance with the basic provisions provided in the *International One- and Two-Family Dwelling Code* (ICC, 1998). Those provisions, in turn, are based on the Southern Forest Products Association's *Permanent Wood Foundations Design and Construction Guide* (SPC, 1998). The PWF guide offers design flexibility and thorough technical guidance. Table 4.7 summarizes some basic rules of thumb for design. The steps for using the prescriptive tables are outlined below.

FIGURE 4.9 Preservative-Treated Wood Foundation Walls

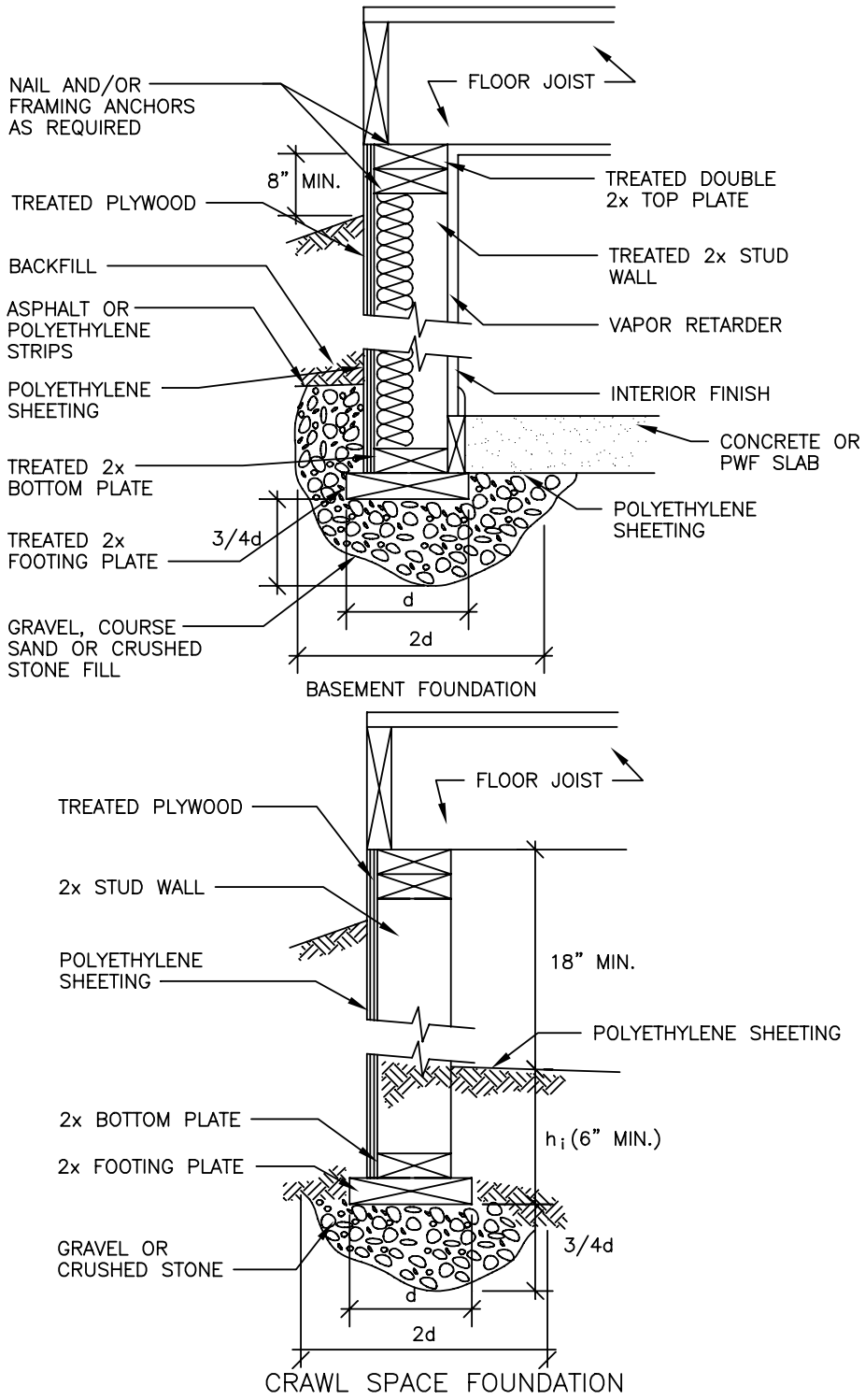


TABLE 4.7 ***Preservative-Treated Wood Foundation Framing***[1]

Maximum Unbalanced Backfill Height (feet)	Nominal Stud Size	Stud Center-to-Center Spacing (inches)
5	2x6	16
6	2x6	12
8	2x8	12

- Connect each stud to top plate with framing anchors when the backfill height is 6 feet or greater.
- Provide full-depth blocking in the outer joist space along the foundation wall when floor joists are oriented parallel to the foundation wall.
- The bottom edge of the foundation studs should bear against a minimum of 2 inches of the perimeter screed board or the basement floor to resist shear forces from the backfill.

Note:
[1]Connection of studs to plates and plates to floor framing is critical to the performance of PWFs. The building code and the *Permanent Wood Foundation Design and Construction Guide* (SPC, 1998) should be carefully consulted with respect to connections.

- Granular (i.e., gravel or crushed rock) footings are sized in accordance with Section 4.4.1. Permanent wood foundations may also be placed on poured concrete footings.
- Footing plate size is determined by the vertical load from the structure on the foundation wall and the size of the permanent wood foundation studs.
- The size and spacing of the wall framing is selected from tables for buildings up to 36 feet wide that support one or two stories above grade.
- APA-rated plywood is selected from tables based on unbalanced backfill height and stud spacing. The plywood must be preservatively treated and rated for below-ground application.
- Drainage systems are selected in accordance with foundation type (e.g., basement or crawl space) and soil type. Foundation wall moisture-proofing is also required (i.e., polyethylene sheeting).

For more information on preservative-treated wood foundations and their specific design and construction, consult the *Permanent Wood Foundations Design and Construction Guide* (SPC, 1998).

4.5.4 Insulating Concrete Form Foundation Walls

Insulating concrete forms (ICFs) have been used in the United States since the 1970s. They provide durable and thermally efficient foundation and above-grade walls at reasonable cost. Insulating concrete forms are constructed of rigid foam plastic, composites of cement and plastic foam insulation or wood chips, or other suitable insulating materials that have the ability to act as forms for cast-in-place concrete walls. The forms are easily placed by hand and remain in place after the concrete is cured to provide added insulation.

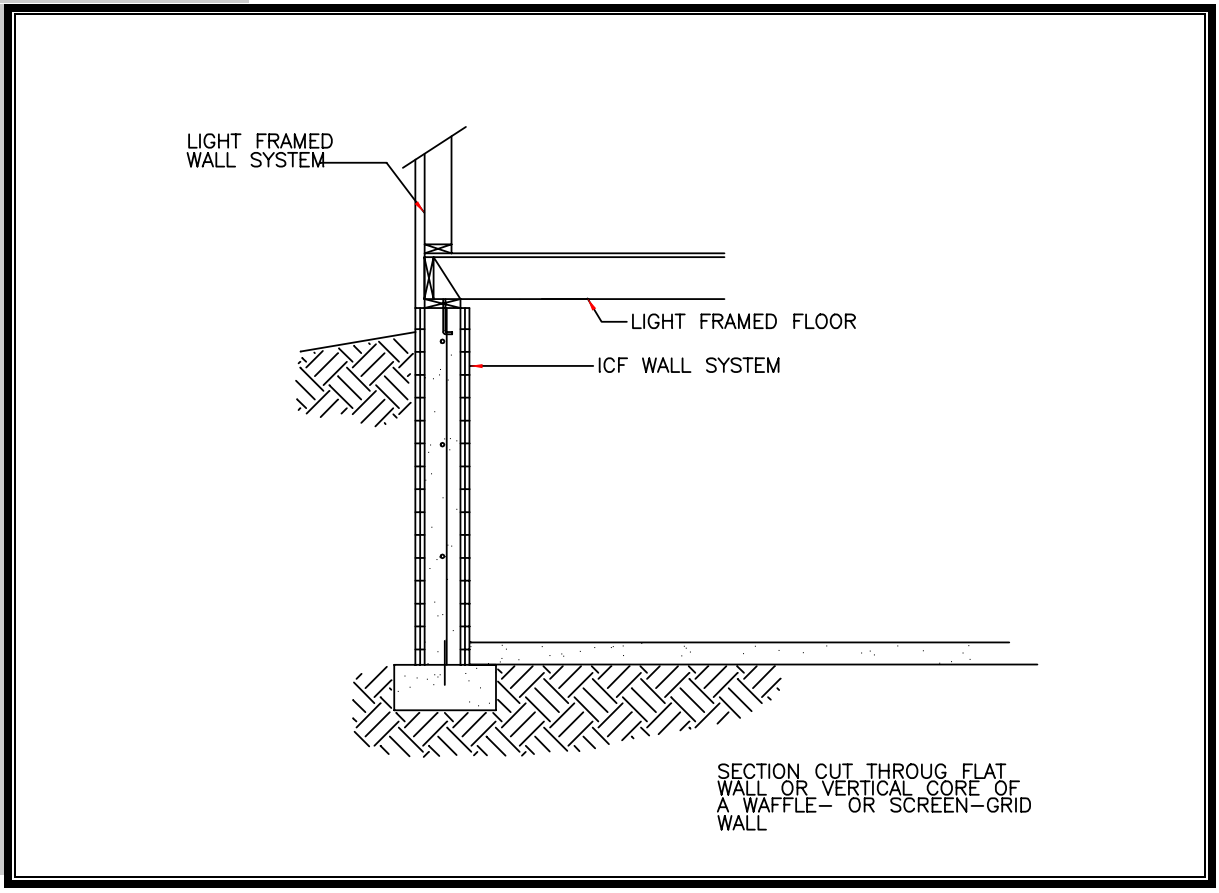
ICF systems are typically categorized with respect to the form of the ICF unit. There are three types of ICF forms: hollow blocks, planks, and panels. The shape of the concrete wall is best visualized with the form stripped away,

exposing the concrete to view. ICF categories based on the resulting nature of the concrete wall are listed below.

- *Flat*. Solid concrete wall of uniform thickness.
- *Post-and-beam*. Concrete frame constructed of vertical and horizontal concrete members with voids between the members created by the form. The spacing of the vertical members may be as great as 8 feet.
- *Screen-grid*. Concrete wall composed of closely spaced vertical and horizontal concrete members with voids between the members created by the form. The wall resembles a thick screen made of concrete.
- *Waffle-grid*. Concrete wall composed of closely space vertical and horizontal concrete members with thin concrete webs filling the space between the members. The wall resembles a large waffle made of concrete.

Foundations may be designed in accordance with the values provided in the most recent national building codes' prescriptive tables (ICC, 1998). Manufacturers also usually provide design and construction information. Insulating concrete form walls are designed by following a procedure similar to that in Section 4.5.1; however, special consideration must be given to the dimensions and shape of an ICF wall that is not a flat concrete wall. Refer to Figure 4.10 for a typical ICF foundation wall detail.

FIGURE 4.10 Insulating Concrete Form Foundation Walls



For more design information, refer to the *Structural Design of Insulating Concrete Form Walls in Residential Construction* (Lemay and Vrankar, 1998). For a prescriptive construction approach, consult the *Prescriptive Method for Insulating Concrete Forms in Residential Construction* (HUD, 1998). These documents can be obtained from the contacts listed in Chapter 1. Manufacturer data should also be consulted.

4.6 Slabs on Grade

The primary objectives of slab-on-grade design are

- to provide a floor surface with adequate capacity to support all applied loads;
- to provide thickened footings for attachment of the above grade structure and for transfer of the load to the earth where required; and to provide a moisture barrier between the earth and the interior of the building.

Many concrete slabs for homes, driveways, garages, and sidewalks are built according to standard thickness recommendations and do not require a specific design unless poor soil conditions, such as expansive clay soils, exist on the site.

For typical loading and soil conditions, floor slabs, driveways, garage floors, and residential sidewalks are built at a nominal 4 inches thick per ACI-302•2.1. Where interior columns and load-bearing walls bear on the slab, the slab is typically thickened and may be nominally reinforced (refer to Section 4.4 for footing design procedures). Monolithic slabs may also have thickened edges that provide a footing for structural loads from exterior load-bearing walls. The thickened edges may or may not be reinforced in standard residential practice.

Slab-on-grade foundations are often placed on 2 to 3 inches of washed gravel or sand and a 6 mil (0.006 inch) polyethylene vapor barrier. This recommended practice prevents moisture in the soil from wicking through the slab. The sand or gravel layer acts primarily as a capillary break to soil moisture transport through the soil. If tied into the foundation drain system, the gravel layer can also help provide drainage.

A slab on grade greater than 10 feet in any dimension will likely experience cracking due to temperature and shrinkage effects that create internal tensile stresses in the concrete. To prevent the cracks from becoming noticeable, the designer usually specifies some reinforcement, such as welded wire fabric (WWF) or a fiber-reinforced concrete mix. The location of cracking may be controlled by placing construction joints in the slab at regular intervals or at strategic locations hidden under partitions or under certain floor finishes (i.e., carpet).

In poor soils where reinforcement is required to increase the slab's flexural capacity, the designer should follow conventional reinforced concrete design methods. The Portland Cement Association (PCA), Wire Reinforcement

Institute (WRI), and U.S. Army Corps of Engineers (COE) espouse three methods for the design of plain or reinforced concrete slabs on grade.

Presented in chart or tabular format, the PCA method selects a slab thickness in accordance with the applied loads and is based on the concept of one equivalent wheel loading at the center of the slab. Structural reinforcement is typically not required; however, a nominal amount of reinforcement is suggested for crack control, shrinkage, and temperature effects.

The WRI method selects a slab thickness in accordance with a discrete-element computer model for the slab. The WRI approach graphically accounts for the relative stiffness between grade support and the concrete slab to determine moments in the slab. The information is presented in the form of design nomographs.

Presented in charts and tabular format, the COE method is based on Westergaard's formulae for edge stresses in a concrete slab and assumes that the unloaded portions of the slab help support the slab portions under direct loading.

For further information on the design procedures for each design method mentioned above and for unique loading conditions, refer to ACI-360, *Design of Slabs on Grade* (ACI, 1998) or the *Design and Construction of Post-Tensioned Slabs on Ground* (PTI, 1996) for expansive soil conditions.

4.7 Pile Foundations

Piles support buildings under a variety of special conditions that make conventional foundation practices impractical or inadvisable. Such conditions include

- weak soils or nonengineered fills that require the use of piles to transfer foundation loads by skin friction or point bearing;
- inland floodplains and coastal flood hazard zones where buildings must be elevated;
- steep or unstable slopes; and
- expansive soils where buildings must be isolated from soil expansion in the "active" surface layer and anchored to stable soil below.

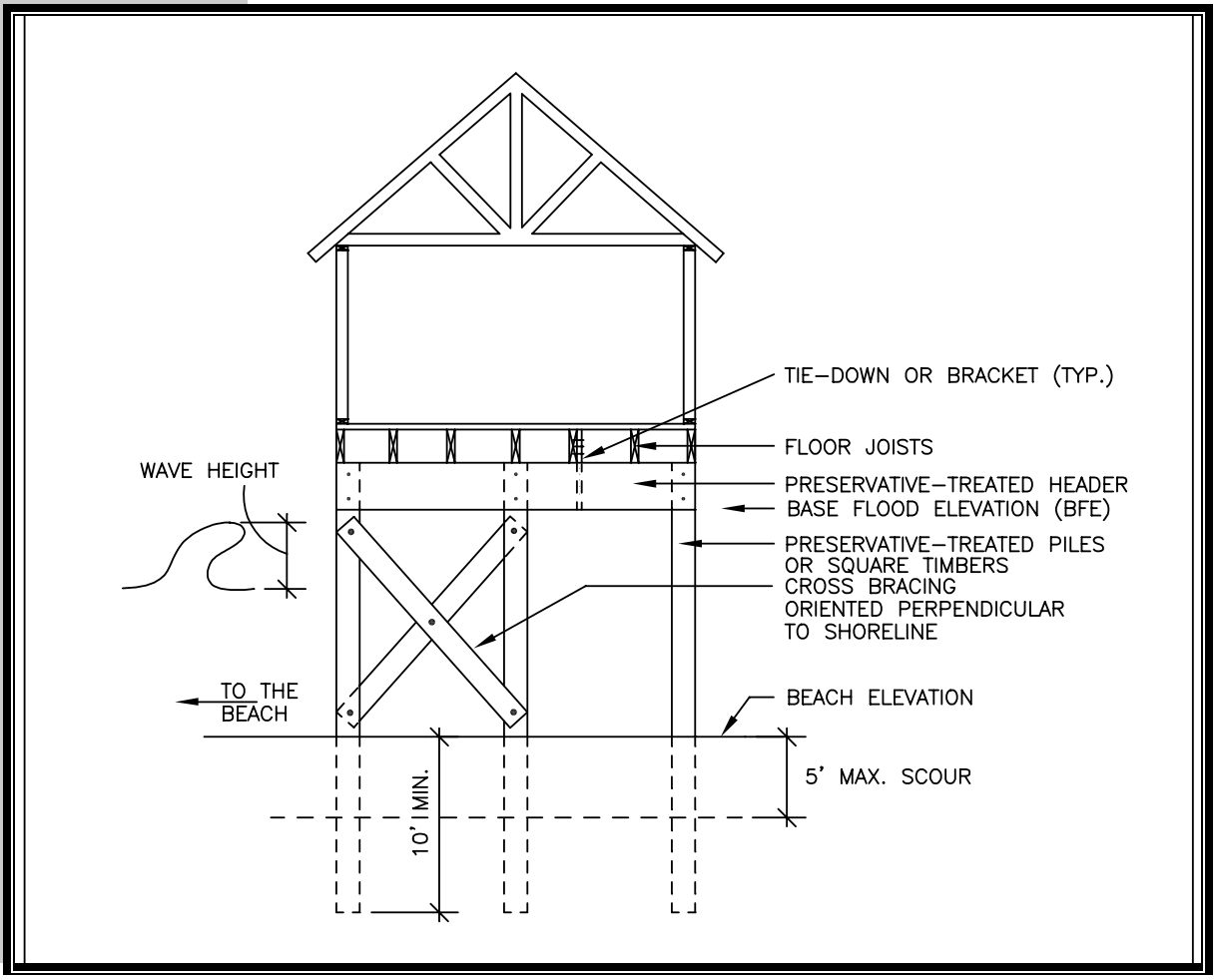
Piles are available in a variety of materials. Preservative-treated timber piles are typically driven into place by a crane with a mechanical or drop hammer (most common in weak soils and coastal construction). Concrete piles or piers are typically cast in place in drilled holes, sometimes with "belled" bases (most common in expansive soils). Steel H-piles or large-diameter pipes are typically driven or vibrated into place with specialized heavy equipment (uncommon in residential construction).

Timber piles are most commonly used in light-frame residential construction. The minimum pile capacity is based on the required foundation loading. Pile capacity is, however, difficult to predict; therefore, only rough estimates of required pile lengths and sizes can be made before installation, particularly when the designer relies only on skin friction to develop capacity in deep, soft soils. For this reason, local successful practice is a primary factor in any pile foundation design such that a pile foundation often can be specified by

experience with little design effort. In other cases, some amount of subsurface exploration (i.e., standard pertrometer test) is advisable to assist in foundation design or, alternatively, to indicate when one or more test piles may be required.

It is rare for pile depth to be greater than 8 or 10 feet except in extremely soft soils, on steeply sloped sites with unstable soils, or in coastal hazard areas (i.e., beachfront property) where significant scour is possible due to storm surge velocity. Under these conditions, depths can easily exceed 10 feet. In coastal high-hazard areas known as "V zones" on flood insurance rating maps (FIRMs), the building must be elevated above the 100-year flood elevation, which is known as the base flood elevation (BFE) and includes an allowance for wave height. As shown in Figure 4.11, treated timber piles are typically used to elevate a structure.

FIGURE 4.11 Basic Coastal Foundation Construction



For additional guidance, the designer is referred to the *Coastal Construction Manual* (FEMA, 1986) and *Pile Buck* (Pile Buck, 1990) but should be prepared to make reasonable design modifications and judgments based on personal experience with and knowledge of pile construction and local conditions. National flood Insurance Program (NFIP) requirements should also be carefully considered by the designer since they may affect the availability of insurance and the premium amount. From a life-safety perspective, pile-supported buildings are

often evacuated during a major hurricane, but flood damage can be substantial if the building is not properly elevated and detailed. In these conditions, the designer must consider several factors, including flood loads, wind loads, scour, breakaway wall and slab construction, corrosion, and other factors. The publications of the Federal Emergency Management Agency (FEMA), Washington, DC, offer design guidance. FEMA is also in the process of updating the *Coastal Construction Manual*.

The habitable portion of buildings in coastal "A zones" (nonvelocity flow) and inland floodplains must be elevated above the BFE, particularly if flood insurance is to be obtained. However, piles are not necessarily the most economical solution. Common solutions include fills to build up the site or the use of crawl space foundations.

For driven timber piles, the capacity of a pile can be roughly estimated from the known hammer weight, drop height, and blow count (blows per foot of penetration) associated with the drop-hammer pile-driving process. Several pile-driving formulas are available; while each formula follows a different format, all share the basic relationship among pile capacity, blow count, penetration, hammer drop height, and hammer weight. The following equation is the widely recognized method first reported in *Engineering News Record* (ENR) and is adequate for typical residential and light-frame commercial applications:

$$P_a = \frac{W_r h}{sF}$$

In the above equation, P_a is the net allowable vertical load capacity, W_r is the hammer ram weight, h is the distance the hammer free falls, s is the pile penetration (set) per blow at the end of driving, and F is the safety factory. The units for s and h must be the same. The value of s may be taken as the inverse of the blow count for the last foot of driving. Using the above equation, a "test" pile may be evaluated to determine the required pile length to obtain adequate bearing.

Alternatively, the designer can specify a required minimum penetration and required number of blows per foot to obtain sufficient bearing capacity by friction. The pile size may be specified as a minimum tip diameter, a minimum butt diameter, or both. The minimum pile butt diameter should not be less than 8 inches; 10- to 12-inch diameters are common. The larger pile diameters may be necessary for unbraced conditions with long unsupported heights.

In hard material or densely compacted sand or hard clay, a typical pile meets "refusal" when the blows per foot become excessive. In such a case, it may be necessary to jet or predrill the pile to a specific depth to meet the minimum embedment and then finish with several hammer blows to ensure that the required capacity is met and the pile properly seated in firm soil.

Jetting is the process of using a water pump, hose, and long pipe to "jet" the tip of the pile into hard-driving ground such as firm sand. Jetting may also be used to adjust the pile vertically to maintain a reasonable tolerance with the building layout dimension.

It is also important to connect or anchor the building properly to pile foundations when severe uplift or lateral load conditions are expected. For standard pile and concrete grade beam construction, the pile is usually extended into the concrete "cap" a few inches or more. The connection requirements of the

National Design Specification for Wood Construction (NDS, 1997) should be carefully followed for these "heavy duty" connections. Such connections are not specifically addressed in Chapter 7, although much of the information is applicable.

4.8 Frost Protection

The objective of frost protection in foundation design is to prevent damage to the structure from frost action (i.e., heaving and thaw weakening) in frost-susceptible soils.

4.8.1 Conventional Methods

In northern U.S. climates, builders and designers mitigate the effects of frost heave by constructing homes with perimeter footings that extend below a locally prescribed frost depth. Other construction methods include

- piles or caissons extending below the seasonal frost line;
- mat or reinforced structural slab foundations that resist differential heave;
- nonfrost-susceptible fills and drainage; and
- adjustable foundation supports.

The local building department typically sets required frost depths. Often, the depths are highly conservative in accordance with frost depths experienced in applications not relevant to residential foundations. The local design frost depth can vary significantly from that required by actual climate, soil, and application conditions. One exception occurs in Alaska, where it is common to specify different frost depths for "warm," "cold," and "interior" foundations. For homes in the Anchorage, Alaska, area, the perimeter foundation is generally classified as warm, with a required depth of 4 or 5 feet. Interior footings may be required to be 8 inches deep. On the other hand, "cold" foundations, including outside columns, may be required to be as much as 10 feet deep. In the contiguous 48 states, depths for footings range from a minimum 12 inches in the South to as much as 6 feet in some northern localities.

Based on the air-freezing index, Table 4.8 presents minimum "safe" frost depths for residential foundations. Figure 4.12 depicts the air-freezing index, a climate index closely associated with ground freezing depth. The most frost-susceptible soils are silty soils or mixtures that contain a large fraction of silt-sized particles. Generally, soils or fill materials with less than 6 percent fines (as measured by a #200 sieve) are considered nonfrost-susceptible. Proper surface water and foundation drainage are also important factors where frost heave is a concern. The designer should recognize that many soils may not be frost-susceptible in their natural state (i.e., sand, gravel, or other well-drained soils that are typically low in moisture content). However, for those that are frost-susceptible, the consequences can be significant and costly if not properly considered in the foundation design.

TABLE 4.8 Minimum Frost Depths for Residential Footings[1,2]

Air-Freezing Index (°F-Days)	Footing Depth (inches)
250 or less	12
500	18
1,000	24
2,000	36
3,000	48
4,000	60

Notes:
[1]Interpolation is permissible.
[2]The values do not apply to mountainous terrain or to Alaska.

4.8.2 Frost-Protected Shallow Foundations

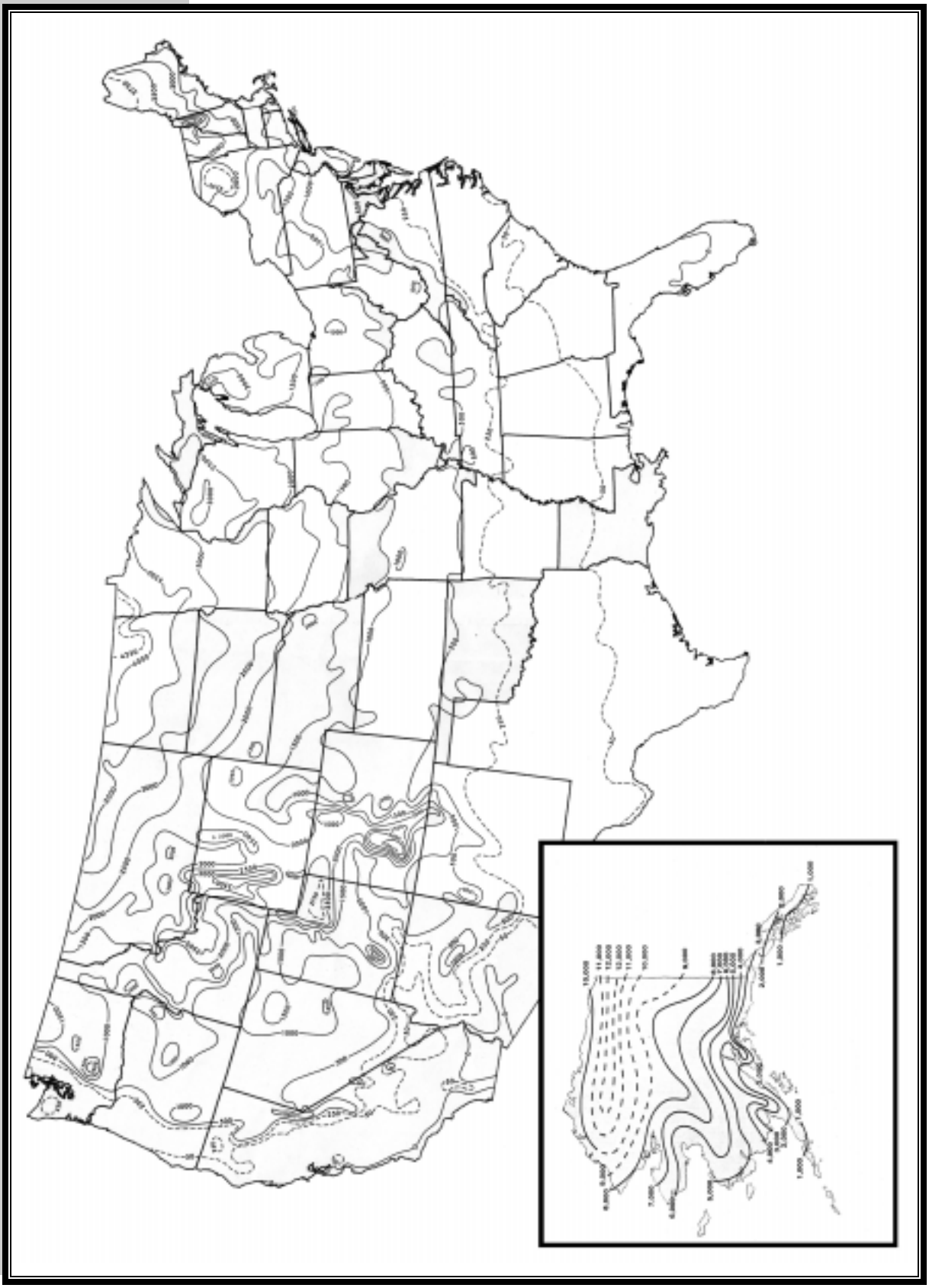
A frost-protected shallow foundation (FPSF) is a practical alternative to deeper foundations in cold regions characterized by seasonal ground freezing and the potential for frost heave. Figure 4.13 illustrates several FPSF applications. FPSFs are best suited to slab-on-grade homes on relatively flat sites. The FPSF method may, however, be used effectively with walkout basements by insulating the foundation on the downhill side of the house, thus eliminating the need for a stepped footing

An FPSF is constructed by using strategically placed vertical and horizontal insulation to insulate the footings around the building, thereby allowing foundation depths as shallow as 12 inches in very cold climates. The frost-protected shallow foundation technology recognizes earth as a heat source that repels frost. Heat input to the ground from buildings therefore contributes to the thermal environment around the foundation.

The thickness of the insulation and the horizontal distance that the insulation must extend away from the building depends primarily on the climate. In less severe cold climates, horizontal insulation is not necessary. Other factors such as soil thermal conductivity, soil moisture content, and the internal temperature of a building are also important. Current design and construction guidelines are based on reasonable “worst-case” conditions.

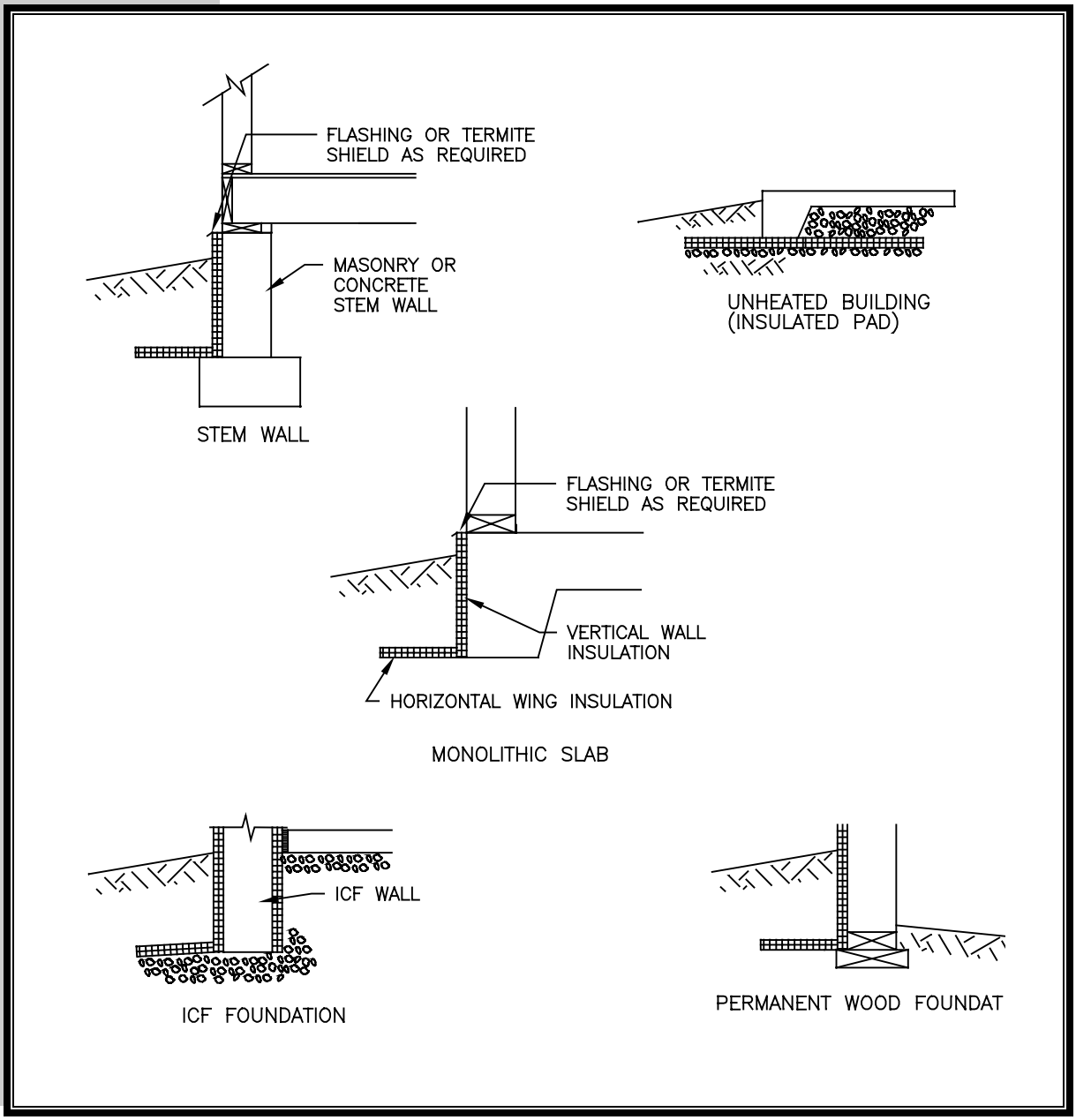
After more than 40 years of use in the Scandinavian countries, FPSFs are now recognized in the prescriptive requirements of the *International One- and Two- Family Dwelling Code* (ICC, 1998) and the 1995 edition. However, the code places limits on the use of foam plastic below grade in areas of noticeably high termite infestation probability. In those areas termite barriers or other details must be incorporated into the design to block “hidden” pathways leading from the soil into the structure between the foam insulation and the foundation wall. The exception to the code limit occurs when termite-resistant materials (i.e., concrete, steel, or preservative-treated wood) are specified for a home’s structural members.

FIGURE 4.12 *Air-Freezing Index Map (100-Year Return Period)*



The complete design procedure for FPSFs is detailed in *Frost Protected Shallow Foundations in Residential Construction, Second Edition* (NAHB Research Center, Inc., 1996). The first edition of this guide is available from the U.S. Department of Housing and Urban Development. Either version provides useful construction details and guidelines for determining the amount (thickness) of insulation required for a given climate or application. Acceptable insulation materials include expanded and extruded polystyrenes, although adjusted insulation values are provided for below-ground use. The American Society of Civil Engineers (ASCE) is currently developing a standard for FPSF design and construction based on the resources mentioned above.

FIGURE 4.13 Frost-Protected Shallow Foundation Applications



4.8.3 Permafrost

Design of residential foundations on permafrost is beyond the scope of this guide. The designer is cautioned that the thawing of permafrost due to a building's thermal effect on a site can quickly undermine a structure. It is critical that the presence of permafrost is properly identified through subsoil exploration. Several effective design approaches are available for building on permafrost. Refer to *Construction in Cold Regions: A Guide for Planners, Engineers, Contractors, and Managers* (McFadden and Bennett, 1991). Permafrost is not a concern in the lower 48 states of the United States.

4.9 Design Examples

EXAMPLE 4.1 Plain Concrete Footing Design

Given Exterior continuous wall footing supporting an 8-inch-wide concrete foundation wall carrying a 12-foot floor tributary width; the wall supports two floor levels each with the same tributary width

Design Loads

Live load	0.75 [(12 ft)(40 psf) +(12 ft)(30 psf)] = 630 plf	(Table 3.1)
Dead load	(12 ft)(10 psf)(2 floors) = 240 plf	(Table 3.2)
Wall dead load	(8 ft)(0.66 ft)(150 pcf) = 800 plf	(Table 3.3)

Footing dead load allowance = 200 plf

Presumptive soil bearing capacity = 1,500 psf (default)
f'_c = 2,000 psi

Find The minimum size of the concrete footing required to support the loads

Solution

1. Determine the required soil bearing area

$$\text{Footing width} = \frac{\text{Design load}}{\text{Presumptive soil bearing}} = \frac{(630\,\text{plf} + 240\,\text{plf} + 800\,\text{plf} + 200\,\text{plf})(1\,\text{ft})}{1{,}500\,\text{psf}} = 1.25\text{ft}$$

The required footing width is equal to

b = 1.25 ft = 15 in ≅ 16 in (standard width of excavation equipment)

2. Preliminary design (rule of thumb method)

Footing projection = 1/2 (16 in. - 8 in.) = 4 in

Required plain concrete footing thickness ≅ 4 in (i.e., no less than the projection)
∴ use minimum 6-inch-thick footing

Footing weight = (1.33 ft)(0.5 ft)(150 pcf) = 100 lb < 200 lb allowance OK

3. Consider design options

- Use 6-inch x16-inch plain wall concrete footing

✔ Design plain concrete footing to check rule of thumb for illustrative purposes only

4. Design a plain concrete footing

(a) Determine soil pressure based on factored loads

$$q_s = \frac{P_u}{A_{footing}} = \frac{(1.2)(240\,plf + 800plf + 200plf) + (1.6)(630plf)}{(1.33\,ft)(1\ ft)} = 1{,}877\ psf$$

(b) Determine thickness of footing based on moment at the face of the wall

$$M_u = \frac{q_s \ell}{8}(b - T)^2$$

$$= \frac{(1{,}877\ psf)(1\ ft)}{8}(1.33\,ft - 0.66\ ft)^2 = 105\,ft - lb/lf$$

$$\phi M_n = 5\sqrt{f'_c}\,S = 5\sqrt{2{,}000\ psi}\,\frac{b\ t^2}{6}$$

$$\phi M_n \geq M_u$$

$$(105\,ft - lb/lf)(12\,in/ft) \geq (0.65)(5)\left(\sqrt{2{,}000\ psi}\right)\left(\frac{(12\,in)\ t^2}{6}\right)$$

t = 2.1 in

(c) Determine footing thickness based on one-way (beam) shear

$$\phi V_c = \phi \frac{4}{3}\sqrt{f'_c}\,\ell t$$

$$= 0.65\left(\frac{4}{3}\right)\sqrt{2{,}000psi}\ (12\,in)(t)$$

$$V_u = (q_s \ell)(0.5(b - T) - t)$$

$$= (1{,}849\ psf)(1\,ft)(0.5(1.33\,ft - 0.66\,ft) - t)$$

$$\phi V_c \geq V_u$$

$$0.65\left(\frac{4}{3}\right)\sqrt{2{,}000\ psi}\ (12\,in)(t) = (1{,}877\ psf)(1\,ft)(0.5(1.33\,ft - 0.66\,ft) - t)$$

t = 0.27 ft = 3.2 in

Therefore, shear in the footing governs the footing thickness

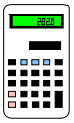
Conclusion

The calculations yield a footing thickness of 3.2 inches. In accordance with ACI-318•22.4.8, two additional inches must be added, resulting in a footing thickness of 5.2 inches. However, in accordance with ACI-318•22.7.4, plain concrete footings may not have a thickness less than 8 inches. A 6-inch-thick plain concrete footing has a history of adequate performance in residential construction and exceeds the calculated thickness requirement. Therefore, use a 6-inch-thick by 16-inch-wide wall footing

In high-hazard seismic areas, a nominal footing reinforcement should be considered (i.e., one No. 4 bar longitudinally). However, longitudinal reinforcement at the top and bottom of the foundation wall provides greater strength against differential soil movement in a severe seismic event, particularly on sites with soft soils.

It is also worthy to note that use of the ACI-318 load combinations in lieu of those provided in Chapter 3 for strength design would have resulted in a calculated footing thickness of 3.2 inches instead of 3.1 inches as governed by flexure. This is a negligible difference for practical purposes.

EXAMPLE 4.2 Reinforced Footing Design



Given Interior footing supporting a steel pipe column (3.5 in x 3.5 in bearing) carrying a 12-ft x 12-ft floor tributary area

Service Loads

Live load (12 ft)(12 ft)(40 psf) = 5,760 lb
Dead load (12 ft)(12 ft)(10 psf) = 1,440 lb
Footing and column dead load = 300 lb (allowance)

Presumptive soil bearing capacity = 1,500 psf (default)
f'_c = 2,500 psi, f_y = 60,000 psi

Find The minimum size of the concrete footing required to support the loads.

Solution

1. Determine the required soil bearing area

$$\text{Area reqd} = \frac{\text{Service load}}{\text{Presumptive soil bearing}} = \frac{(5{,}760\,\text{lb} + 1{,}440\,\text{lb} + 300\,\text{lb})}{1{,}500\,\text{psf}} = 5\,\text{ft}^2$$

Assume a square footing

$$b = \sqrt{5\,\text{ft}^2} = 2.2\,\text{ft} = 26\,\text{in}$$

2. Preliminary design (rule of thumb method)

Footing projection = 1/2 (26 in - 3.5 in) = 11.25 in

$\therefore$ Required plain concrete footing thickness $\cong$ 12 in

Footing weight = (5 ft^2)(1 ft)(150 pcf) = 750 lb > 300 lb allowance

$\therefore$ Recalculation yields a 28-in x 28-in footing.

3. Consider design options

- use 12-in x 28-in x28-in plain concrete footing (5 ft^3 of concrete per footing $);
- reduce floor column spacing (more but smaller footings, perhaps smaller floor beams, more labor)
- test soil bearing to see if higher bearing value is feasible (uncertain benefits, but potentially large, i.e., one-half reduction in plain concrete footing size);
- design a plain concrete footing to determine if a thinner footing is feasible; or

✔ design thinner, reinforced concrete footing (trade-off among concrete, rebar, and labor)

4. Design a reinforced concrete footing

Given Square footing, 28 in x 28 in
f'_c= 2,500 psi concrete; 60,000 psi steel

Find Footing thickness and reinforcement

(a) Select trial footing thickness, rebar size, and placement

t = 6 in
c = 3 in
d_b = 0.5 in (No. 4 rebar)

(b) Calculate the distance from extreme compression fiber to centroid of reinforcement d

$d = t\text{-}c\text{-}0.5d_b$
= 6 in – 3 in - 0.5 (0.5 in)
= 2.75 in

(c) Determine soil pressure based on factored loads

$$q_s = \frac{P_u}{A_{footing}} = \frac{(1.2)(1{,}440\,\text{lb} + 300\,\text{lb}) + (1.6)(5{,}760\,\text{lb})}{5\,\text{ft}^2} = 2{,}261\,\text{psf}$$

(d) Check one-way (beam) shear in footing for trial footing thickness

$$\phi V_c = \phi 2\sqrt{f'_c}\,bd$$
$$= 0.85\,(2)\sqrt{2{,}500\,\text{psi}}\,(28\,\text{in})(2.75\,\text{in}) = 6{,}545\,\text{lbs}$$
$$V_u = \left(\frac{P_u}{b}\right)(0.5\,(b - T) - d) =$$
$$= \left(\frac{11{,}304\,\text{lbs}}{28\,\text{in}}\right)(0.5\,(28\,\text{in} - 3.5\,\text{in}) - 2.75\text{in}) = 3{,}835\,\text{lbs}$$
$\phi V_c >> V_u$ OK

(e) Check two-way (punching) shear in trial footing

$$\phi V_c = \phi 4\sqrt{f'_c}\,b_o d$$
$$= (0.85)\,(4)\sqrt{2{,}500}\ \text{psi}\,(4(3.5\text{in} + 2.75\text{in}))(2.75\text{in}) = 11{,}688\text{lbs}$$
$$V_u = \left(\frac{P_u}{b^2}\right)\left(b^2 - (T + d)^2\right)$$
$$= \frac{11{,}304\ \text{lbs}}{(28\ \text{in})^2}\left((28\ \text{in})^2 - (3.5\ \text{in} + 2.75\ \text{in})^2\right) = 10{,}741\ \text{lbs}$$
$\phi V_c > V_u$ OK

(f) Determine reinforcement required for footing based on critical moment at edge of column

$$M_u = q_s b(0.5)(0.5(l - T))^2$$

$$= (2{,}261\,\text{psf})\left(\frac{28\,\text{in}}{12\ \text{in/ft}}\right)(0.5)\left(0.5\left(\frac{28\ \text{in}}{12\ \text{in/ft}} - \frac{3.5\ \text{in}}{12\ \text{in/ft}}\right)\right)^2 = 2{,}749\ \text{ft} - \text{lbs}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{(2{,}749\,\text{ft} - \text{lbs})(12\,\text{in/ft})}{(0.9)(28\,\text{in})(2.75\,\text{in})^2} = 173\,\text{psi}$$

$$\rho = \left(\frac{0.85 f'_c}{f_y}\right)\left(1 - \sqrt{\frac{2R_n}{0.85 f'_c}}\right)$$

$$= \left(\frac{0.85(2{,}500\,\text{psi})}{60{,}000\,\text{psi}}\right)\left(1 - \sqrt{\frac{(2)(146\,\text{psi})}{0.85(2{,}500\,\text{psi})}}\right) = 0.022$$

$$\rho_{(\text{gross})} = \frac{d}{t}\rho = \left(\frac{2.75\,\text{in}}{6\,\text{in}}\right)(0.022) = 0.010$$

$\rho_{\text{gross}} \geq \rho_{\text{min}} = 0.0018$ OK

$A_s = \rho b d = 0.010\ (28\ \text{in})(2.75\ \text{in}) = 0.77\,\text{in}^2$

Use four No. 4 bars where $A_s = 4(0.2\ \text{in}^2) = 0.8\ \text{in}^2 \geq 0.77\ \text{in}^2$ OK

Conclusion

Use minimum 28-in x 28-in x 6-in footing with four No. 4 bars or three No. 5 bars each way in footing.

f'_c = 2,500 psi minimum (concrete)
f_y = 60,000 psi minimum (steel reinforcing bar)

EXAMPLE 4.3 *Plain Concrete Foundation Wall Design*

Given

Design loads

Snow load (S)	=	280 plf
Live load (L)	=	650 plf
Dead load (D)	=	450 plf
Moment at top	=	0
Concrete weight	=	150 pcf
Backfill material	=	45 pcf
f'_c = 3,000 psi		
Wall thickness	=	8 in
Wall height	=	8 ft
Unbalanced backfill height	=	7 ft

Assume axial load is in middle one-third of wall

Find Verify that an 8-inch-thick plain concrete wall is adequate for the following load combinations from Chapter 3 (Table 3.1)

- 1.2D + 1.6H
- 1.2D + 1.6H + 1.6L + 0.5 (L_r + S)
- 1.2D + 1.6H = 1.6 (L_r + S) + 0.5L

Only the first load combination will be evaluated since it can be shown to govern the wall design.

Solution

1. Determine loads

Equivalent fluid density of backfill soil

Silty clay: w = 100 pcf, K_a = 0.45 (see Section 3.5)

$q = K_a w = (0.45)(100\ \text{pcf}) = 45\ \text{pcf}$

Total lateral earth load

$$H = \frac{1}{2} q l^2 = \frac{1}{2}(45\text{pcf})(7\text{ft})^2 = 1{,}103\ \text{plf}$$

$$X_1 = \frac{1}{3} l = \frac{1}{3}(7\ \text{ft}) = 2.33\ \text{ft}$$

Maximum shear occurs at bottom of wall (see Figure A.1 of Appendix A)

$$V_{bottom} = V_1 = \frac{1}{2} q h^2 \left(1 - \frac{h}{3L}\right) = \frac{1}{2}(45\ \text{pcf})(7\ \text{ft})^2 \left(1 - \frac{7\ \text{ft}}{3(8\ \text{ft})}\right) = 781\ \text{plf}$$

Maximum moment and its location

$$x = h - \sqrt{h^2 - \frac{2V_1}{q}}$$

$$= 7\,\text{ft} - \sqrt{(7\,\text{ft})^2 - \frac{2(781\,\text{plf})}{45\,\text{pcf}}}$$

= 3.2 ft from base of wall or 4.8 ft from top of wall

$$M_{max}\ (\text{at } x = 3.2\text{ ft}) = V_1 x - \frac{1}{2} qhx^2 + \frac{1}{6} qx^3$$

$$= (781\text{ plf})(3.2\text{ ft}) - \frac{1}{2}(45\text{ pcf})(7\text{ ft})(3.2\text{ ft})^2 + \frac{1}{6}(45\text{ pcf})(3.2\text{ ft})^3$$

= 1,132 ft-lb/ℓf

2. Check shear capacity

(a) Factored shear load

$V_u = 1.6\ V_{bottom}$
$= 1.6\ (781\text{ plf}) = 1,250\text{ plf}$

(b) Factored shear resistance

$$\phi V_n = \phi \frac{4}{3}\sqrt{f'_c}\,bh$$

$$= (0.65)\left(\frac{4}{3}\right)\sqrt{3{,}000\text{psi}}\,(8\,\text{in})(12\,\text{in}) = 4{,}557\,\text{plf}$$

(c) Check $\phi V_n \geq V_u$

4,557 plf >> 1,250 plf OK

Shear is definitely not a factor in this case. Future designs of a similar nature may be based on this experience as "OK by inspection."

3. Check combined bending and axial load capacity

(a) Factored loads

$M_u = 1.6\ M_{max} = 1.6\ (1{,}132\text{ ft-lb/lf}) = 1{,}811\text{ ft-lb/lf}$
$P_u = 1.2\ D$
$D_{structure} = 450\text{ plf (given)}$

$$D_{concrete@x} = (150\text{ plf})\left(\frac{8\text{ in}}{12\text{ in/ft}}\right)(8\text{ ft} - 3.23\text{ ft}) = 480\,\text{plf}$$

$D = 450\text{ plf} + 480\text{ plf} = 930\text{ plf}$
$P_u = 1.2\ (930\text{ plf}) = 1{,}116\text{ plf}$

(b) Determine M_n, M_{min}, P_u

$M_n = 0.85\ f'_c S$

$S = \frac{1}{6}bd^2 = \left(\frac{1}{6}\right)(12\text{ in})(8\text{ in})^2 = 128\text{ in}^3/\text{lf}$

$M_n = 0.85\ (3{,}000\text{ psi})(128\text{ in}^3/\text{lf}) = 326{,}400\text{ in-lb/lf} = 27{,}200\text{ ft-lb/lf}$

$M_{min} = 0.1hP_u = 0.1\left(\frac{8\text{ in}}{12\text{ in/lf}}\right)(1{,}112\text{ plf}) = 74\text{ ft-lb/lf}$

$M_u > M_{min}$ OK

$$P_n = 0.6f'_c\left[1-\left(\frac{L}{32h}\right)^2\right]A_g$$

$$= 0.6(3{,}000\text{psi})\left[1-\left(\frac{(8\text{ ft})\left(\frac{12\text{ in}}{\text{ft}}\right)}{32\ (8\text{ in})}\right)^2\right](8\text{ in})(12\text{ in}) = 148{,}500\text{ plf}$$

(c) Check combined bending and axial stress equations

Compression $\quad \frac{P_u}{\phi P_n}+\frac{M_u}{\phi M_n} \le 1$

$$\frac{1{,}116\text{ plf}}{(0.65)(148{,}500\text{ plf})}+\frac{1{,}811\text{ ft-lb/lf}}{(0.65)(27{,}200\text{ ft-lb/lf})} \le 1$$

$0.11 \le 1$ OK

Tension $\quad \frac{M_u}{S}-\frac{P_u}{A_g} \le \phi 5\sqrt{f'_c}$

$$\frac{1{,}811\text{ ft-lb/lf}\ (12\text{ in/ft})}{128\text{ in}^3/\text{lf}}-\frac{1{,}116\text{ plf}}{(8\text{ in})(12\text{ in})} \le (0.65)(5)\sqrt{3{,}000\text{ psi}}$$

$158 \le 178$ OK

∴ No reinforcement required

4. Check deflection at mid-span (see Figure A.1 in Appendix A)

$$\rho_{max} \cong \frac{qL^3}{E_c I_g}\left[\frac{hL}{128} - \frac{L^2}{960} - \frac{h^2}{48} + \frac{h^3}{144L}\right]$$

$$= \frac{(45\,\text{pcf})(8\,\text{ft})^3}{(3{,}122{,}019\,\text{psi})\left(\frac{12\,\text{in}\,(8\,\text{in})^3}{12}\right)}\left[\frac{(7\,\text{ft})(8\,\text{ft})}{128} - \frac{(8\,\text{ft})^2}{960} - \frac{(7\,\text{ft})^2}{48} + \frac{(7\,\text{ft})^3}{144\,(8\,\text{ft})}\right]\left(\frac{1{,}728\,\text{in}^3}{\text{ft}^3}\right)$$

$= 0.009$ in/lf

$$\rho_{all} = \frac{L}{240} = \frac{(8\ \text{ft})(12\ \text{in}/\text{ft})}{240} = 0.4\ \text{in}/\text{lf}$$

$\rho_{max} << \rho_{all}$ OK

Conclusion

An 8-inch-thick plain concrete wall is adequate under the given conditions.

The above analysis was performed for a given wall thickness. The same equations can be used to solve for the minimum wall thickness h that satisfies the requirements for shear, combined bending and axial stress, and deflection. With this approach to the problem, the minimum thickness would be 7.6 inches (controlled by tensile stress under combined bending and axial load).

In the strength-based design approach, the safety margin is related to the use of load and resistance factors. In this problem, the load factor was 1.6 (for a soil load, H) and the resistance factor 0.65 (for tensile bending stress). In terms of a traditional safety factor, an equivalent safety margin is found by 1.6/0.65 = 2.5. It is a fairly conservative safety margin for residential structures and would allow for an equivalent soil fluid density of as much as 113 pcf (45 pcf x 2.5) at the point the concrete tensile capacity based on the minimum concrete compressive strength (as estimated by $5\sqrt{f'_c}$) is realized. This capacity would exceed loads that might be expected should the soil become saturated as would occur under severe flooding on a site that is not well drained.

The use of reinforcement varies widely as an optional enhancement in residential construction to control cracking and provide some nominal strength benefits. If reinforcement is used as a matter of good practice, one No. 4 bar may be placed as much as 8 feet on-center. One horizontal bar may also be placed horizontally at the top of the wall and at mid-height.

EXAMPLE 4.4 *Plain Concrete Wall Interaction Diagram*

Given Construct an interaction diagram for the wall in Design Example 4.3

Wall height = 8 ft
Wall thickness = 8 in
f'_c = 3,000 psi

Solution

1. Determine compression boundary

$$P_n = 0.6 f'_c \left[1-\left(\frac{L}{32h}\right)^2\right] A_g$$

$$= 0.6(3{,}000\text{psi})\left[1-\left(\frac{(8\text{ ft})(12\text{ in/lf})}{32(8\text{ in})}\right)^2\right](8\text{ in})(12\text{ in}) = 148{,}500\text{ plf}$$

$$M_n = 0.85 f'_c S$$

$$= (0.85)(3{,}000\text{psi})\frac{(12\text{ in})(8\text{ in})^2}{6}$$

$$= 326{,}400\text{ in-lb/lf} = 27{,}200\text{ ft-lb/lf}$$

$$A_g = (8\text{ in})(12\text{ in}) = 96\text{ in}^2$$

$$\frac{P_u}{\phi P_n}+\frac{M_u}{\phi M_n} \le 1$$

$$\frac{P_u}{0.65\ (148{,}500\text{ plf})} + \frac{M_u}{0.65\ (27{,}200\text{ ft}-\text{lb/lf})} \le 1$$

$$\frac{P_u}{(96{,}525\text{plf})} + \frac{M_u}{17{,}680\text{ft}-\text{lb/lf}} \le 1$$

$$M_u = \left(1-\frac{P_u}{96{,}525\text{plf}}\right)17{,}680\text{ ft}-\text{lb/lf}$$

$$M_u = 17{,}680\text{ ft}-\text{lb/lf} - 0.18316\ P_u$$

$$P_u = \left(1-\frac{M_u}{17{,}680\text{ ft}-\text{lb/lf}}\right)96{,}525\text{ plf}$$

$$P_u = 96{,}525\text{ plf} - 5.46\ M_u$$

When $P_u = 0$, $M_u = 17{,}680$ ft-lb/lf
When $M_u = 0$, $P_u = 96{,}525$ plf (0, 96.5klf)

2. Determine tension boundary

$$\frac{M_u}{S} - \frac{P_u}{A_g} \le 5\phi\sqrt{f'_c}$$

$$\frac{M_u}{128\text{in}^3} - \frac{P_u}{96\text{in}^2} \le 5(0.65)\sqrt{3{,}000\text{ psi}}$$

$$\frac{M_u}{128\text{in}^3} - \frac{P_u}{96\text{in}^2} \le 178\text{ psi}$$

$$P_u = 96\text{in}^2\left(\frac{M_u}{128\text{in}^3} - 178\text{ psi}\right)$$

$$P_u = 0.75M_u - 17{,}088\text{ plf}$$

When $M_u = 0$; P_u = -17,088plf = - 17.09 klf (-17.09,0)

3. Determine point of intersection of the tensile and compression boundaries

$$P_u = \frac{\phi Mn - 5\phi\sqrt{f'_c}\ S}{\frac{S}{A_g} + \frac{\phi Mn}{\phi Pn}}$$

$$= \frac{(0.65)(27{,}200\,\text{ft}-\text{lb/lf})(12\,\text{in/ft}) - 5(0.65)\sqrt{3{,}000\,\text{psi}}\,(128\,\text{in}^3)}{\frac{128\,\text{in}^3}{96\,\text{in}^2} + \frac{0.65(27{,}200\,\text{ft}-\text{lb/lf})(12\,\text{in/ft})}{96{,}525\,\text{plf}}} = 53{,}627\,\text{plf}$$

$$= 53.63\,\text{klf}$$

$$M_u = \phi M_n\left(1 - \frac{(1{,}000\,\text{lb/kip})P_u}{\phi P_n}\right)$$

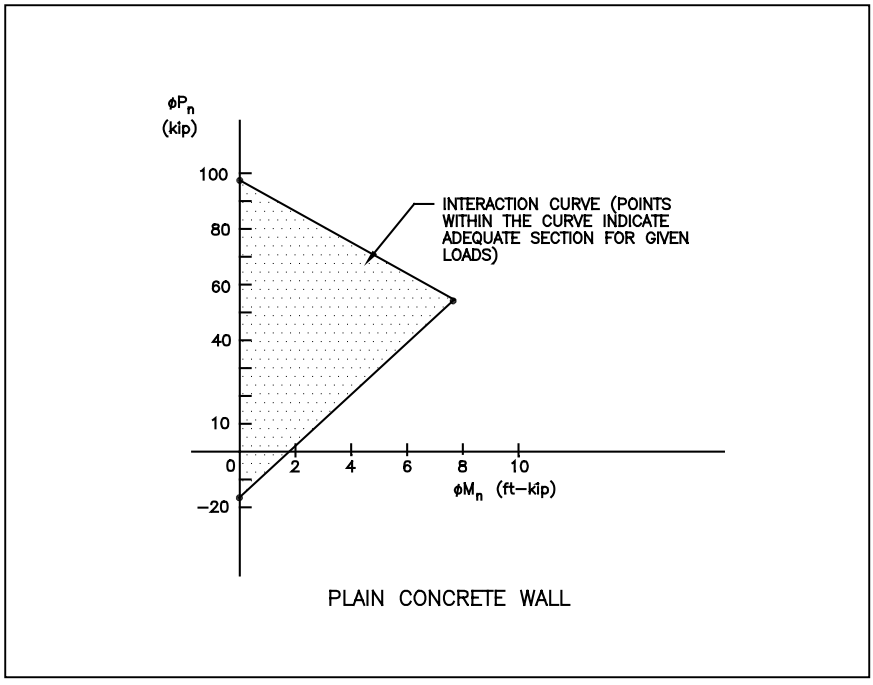
$$= (0.65)(12\,\text{in/ft})(27{,}200\,\text{ft}-\text{lb/lf})\left(1 - \frac{(1{,}000\,\text{lb/kip}\,(53.63))}{96{,}525\,\text{plf}}\right)$$

$$= 94{,}282\,\text{in}-\text{lb/lf} = 7.9\,\text{ft}-\text{kip/lf}$$

Conclusion

Shown below is the interaction diagram for an 8-foot-high, 8-inch-thick plain concrete wall where the concrete compressive strength is 3,000 psi. The interaction diagram uses the points determined in the above steps.

(0, 96.5) from step (1)
(-17.09, 0) from step (2)
(7.9, 53.63) from step (3)



Interaction Diagram

EXAMPLE 4.5 *Moment Magnifier*

Given

Service loads

Live load (L)	= 1,000 plf
Dead load (D)	= 750 plf
Moment at top, (M_{top})	= 0
M_u	= 2,434 ft-lb/lf
Concrete weight	= 150 pcf
Backfill material	= 45 pcf (equivalent fluid density)

f'_c = 3,000 psi
One No. 6 bar at 12 inches on-center (A_s=0.44 in^2)
Nonsway frame

Wall thickness	= 8 in
Wall height	= 10 ft

Assume axial load is in middle one-third of wall

Find The moment magnifier for load combination U = 1.2D + 1.6L (Chapter 3, Table 3.1)

Solution

1. Determine total axial load on wall

P_u = 1.2 D + 1.6L
1.2 (750 plf) + 1.6 (1,000 plf) = 2,500 plf

2. Determine approximate moment magnifier by using the table in Section 4.3.1.3, assuming the axial load is 2,500 plf

		P_u 2,000 lbs	P_u 4,000 lbs
7.5-in-thick wall	10 ft height	1.04	1.09
9.5-in-thick wall	10 ft height	1.00	1.04

For an 8-in-thick wall, 10-ft-high with approximately 3,000 plf factored axial load acting on the wall, the magnifier through interpolation is

$$\delta_{ns} \cong 1.04$$

The objective has been met; however, the detailed calculations to determine the moment magnifier are shown below for comparison purposes.

3. Calculate the moment magnifier

$$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{3{,}000\ \text{psi}} = 3{,}122{,}019\ \text{psi}$$

$$\beta_d = \frac{P_{u,dead}}{P_u} = \frac{(1.2)(750\ \text{plf})}{1.2\,(750\ \text{plf}) + 1.6\,(1{,}000\ \text{plf})} = 0.36$$

$$\rho = \frac{A_s}{A_g} = \left(\frac{0.44\ \text{in}^2}{(8\ \text{in})(12\ \text{in})}\right) = 0.0046$$ [one No. 6 at 12 inches; A_s = 0.44 in^2 OK]

$\beta = 0.9 + 0.5\,\beta_d^2 - 12\rho \geq 1$
$= 0.9 + 0.5\,(0.36)^2 - 12\,(0.0046) = 0.91 < 1$
$= 1$ (governs)

$M_{2,min} = P_u (0.6 + 0.03h)$

$= (2{,}500 \text{ plf})(0.6 + 0.03 (8 \text{ in})) = 2{,}100 \text{ in-lb/lf}$

$M_u = 2{,}434 \text{ ft-lb/lf} \leftarrow \text{Governs}$

$$e = \frac{M_2}{P_u} = \frac{2{,}434 \text{ ft} - \text{lb/lf}}{(2{,}500 \text{ plf})} = 0.89 \text{ ft} = 10.7 \text{ in}$$

$$EI = \frac{0.4E_c I_g}{\beta} \geq \frac{E_c I_g (0.5 - \frac{e}{h})}{\beta} \geq \frac{0.1E_c I_g}{\beta}$$

$$EI = \frac{(3{,}122{,}019 \text{ psi})\left(\frac{(12 \text{ in})(8 \text{ in})^3}{12}\right)\left(0.5 - \frac{10.7 \text{ in}}{8 \text{ in}}\right)}{1} = -1.3 \times 10^9 \text{ lb} - \text{in}^2/\text{lf}$$

$$EI_{max} = \frac{0.4(3{,}122{,}019 \text{ psi})\left(\frac{(12 \text{ in})(8 \text{ in})^3}{12}\right)}{1} = 6.4 \times 10^8 \text{ lb} - \text{in}^2/\text{lf}$$

$$EI_{min} = \frac{0.1(3{,}122{,}019 \text{ psi})\left(\frac{(12 \text{ in})(8 \text{ in})^3}{12}\right)}{1} = 1.6 \times 10^8 \text{ lb} - \text{in}^2/\text{lf} \text{ (governs)}$$

$C_m = 1$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} = \frac{\pi^2 (1.6 \times 10^8 \text{ lb} - \text{in}^2/\text{lf})}{(1(10 \text{ ft})(12 \text{ in/ft}))^2} = 109{,}662 \text{ plf}$$

$$\delta_{ns} = \frac{C_m}{1 - \left(P_u / 0.75 P_c\right)} \geq 1.0$$

$$= \frac{1}{1 - \left(\frac{2{,}500 \text{plf}}{0.75 (109{,}662 \text{plf})}\right)} = 1.03 \geq 1$$

$\delta_{ns} = 1.03$

Conclusion

The moment magnifier by the approximation method is 1.04. It is slightly conservative but saves time in calculation. Through calculation, a slight efficiency is achieved and the calculated moment magnifier is 1.03.

EXAMPLE 4.6 *Reinforced Concrete Foundation Wall Design*

Given

Service loads

Live load (L)	= 1000 plf
Dead load (D)	= 750 plf
Moment at top	= 0
Concrete weight	= 150 pcf
Backfill material	= 60 pcf (equivalent fluid density)
Wall thickness	= 8 in
Wall height	= 10 ft
Unbalanced backfill height	= 8 ft

f'_c=3,000 psi, f_y=60,000 psi
Assume axial load is in middle one-third of wall

Find If one No. 5 bar at 24 inches on-center vertically is adequate for the load combination, U = 1.2D + 1.6H + 1.6L (Chapter 3, Table 3.1) when rebar is placed 3 inches from the outer face of wall (d=5 in)

Solution

1. Determine loads

Total lateral earth load

$$H = \frac{1}{2}ql^2 = \frac{1}{2}(60\,\text{pcf})(8\,\text{ft})^2 = 1{,}920\,\text{plf}$$

$$X = \frac{1}{3}l = \frac{1}{3}(8\text{ ft}) = 2.67\text{ ft}$$

Maximum shear occurs at bottom of wall

$$\Sigma M_{top} = 0$$

$$V_{bottom} = \frac{H(L-x)}{L} = \frac{(1{,}920\,\text{plf})(10\text{ ft} - 2.67\text{ ft})}{10\text{ ft}} = 1{,}408\text{ plf}$$

Maximum moment and its location

$$X_{max} = \frac{ql - \sqrt{q^2l^2 - 2qV_{bottom}}}{q}$$

$$= \frac{(60\,\text{pcf})(8\,\text{ft}) - \sqrt{(60\,\text{pcf})^2(8\,\text{ft})^2 - 2(60\,\text{pcf})(1{,}408\,\text{plf})}}{60\,\text{pcf}}$$

X_{max} = 3.87 ft from base of wall or 6.13 ft from top of wall

$$M_{max} = \frac{-qlx_{max}^2}{2} + \frac{qx_{max}^3}{6} + V_{bottom}(x_{max})$$

$$= \frac{-(60\,\text{pcf})(8\,\text{ft})(3.87\,\text{ft})^2}{2} + \frac{(60\text{pcf})(3.87\,\text{ft})^3}{6} + (1{,}408\,\text{plf})(3.87\,\text{ft})$$

= 2,434 ft-lb/lf

2. Check shear capacity assuming no shear reinforcement is required (V_s=0)

(a) Factored shear load

$$V_u = 1.6\ V_{bottom}$$
$$= 1.6\ (1{,}408\ \text{plf}) = 2{,}253\ \text{plf}$$

(b) Factored shear resistance

$$\phi V_n = \phi\ (V_c + V_s)$$
$$= \phi\ (2) \sqrt{f'_c}\ b_w d$$
$$= (0.85)\ (2)\ \sqrt{3{,}000\ \text{psi}}\ (12\ \text{in})\ (5\ \text{in}) = 5{,}587\ \text{plf}$$

(c) Check $\phi V_n \geq V_u$

5,587 plf >> 2,253 plf OK

Shear is definitely not a factor in this case. Future designs of a similar nature may be based on this experience as "OK by inspection"

3. Determine slenderness

All four foundation walls are concrete with few openings; therefore, the system is a nonsway frame. This is a standard assumption for residential concrete foundation walls.

Slenderness $$r = \sqrt{\frac{I_g}{A_g}} = \sqrt{\frac{\left(\frac{1}{12}\right)(12\ \text{in})\ (8\ \text{in})^3}{(8\ \text{in})(12\ \text{in})}} = 2.31$$

$$\frac{kl_u}{r} < 34$$

$$\frac{(1)(8\ \text{in})(12\ \text{in})}{2.31} = 41.6 \geq 34 \quad \therefore \text{ Use moment magnifier method}$$

4. Determine the magnified moment using the moment magnifier method

$$P_u = 1.2D + 1.6L = 1.2\ (750\ \text{plf}) + 1.6\ (1{,}000\ \text{plf}) = 2{,}500\ \text{plf}$$

Using the approximated moment magnifiers in Table 4.4, the moment magnifier from the table for a 7.5-inch-thick wall, 10-feet-high is between 1.04 and 1.09. For a 9.5-inch-thick wall, the values are between 1 and 1.04.

Through interpolation, δ = 1.04 for a 2,500 plf axial load.

5. Check pure bending

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.155\,in^2)(60{,}000\,psi)}{0.85\ (3{,}000\,psi)(12\,in)} = 0.304$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$= 0.9\ (0.155\ in^2)(60{,}000\ psi)\left(5\ in - \frac{0.304\,in}{2}\right)$$

$$= 40{,}577\ in\text{-}lb/lf = 3{,}381\ ft\text{-}lb/lf$$

$\phi P_n = 0$

$M_u = 2{,}434$ ft-lb/lf from step (1)

$\delta M_u = 1.04\ (2{,}434\ ft\text{-}lb/lf) = 2{,}531$ ft-lb/lf

By inspection of the interaction diagram in Example 4.6, one No. 5 at 24 inches on center is OK since $\delta M_u P_u$ is contained within the interaction curve. See Example 4.6 to construct an interaction diagram.

6. Check deflection

$$\rho_{max} = \left[-\frac{q(x - L + l)^5}{120} + \frac{ql^3x^3}{36L} + \frac{ql^5x}{120L} - \frac{ql^3Lx}{36} \right] \Big/ E_c I_g$$

$$= \frac{(1728\,in^3)}{ft^3} \left[-\frac{(60\,pcf)(6.13\,ft - 10\,ft + 8\,ft)^5}{120} + \frac{(60\,pcf)(8\,ft)^3(6.13\,ft)^3}{36(10\,ft)} + \frac{(60\,pcf)(8\,ft)^5(6.13\,ft)}{120(10\,ft)} - \frac{(60\,pcf)(8\,ft)^3(10\,ft)(6.13\,ft)}{36} \right] \Big/ (3{,}122{,}019\,psi)\left(\frac{(12\,in)(8\,in)^3}{12}\right)$$

$= 0.025$ in/lf

$$\rho_{all} = \frac{L}{240} = \frac{(10\,ft)(12\,in/ft)}{240} = 0.5\,in/lf$$

$\rho_{max} \ll \rho_{all}$ OK

Conclusion

An 8-inch-thick reinforced concrete wall with one vertical No. 5 bar at 24 inches on-center is adequate for the given loading conditions.

This analysis was performed for a given wall thickness and reinforcement spacing. The same equations can be used to solve for the minimum reinforcement that satisfies the requirements for shear, combined bending and axial stress, and deflection. This approach would be suitable for a computer spreadsheet design aid. A packaged computer software program can also be purchased to perform this function; however, certain limitations may prohibit the designer from using design recommendations given in this guide.

The use of horizontal reinforcement varies widely as an optional enhancement. If horizontal reinforcement is used as a matter of preferred practice to control potential cracking, one No. 4 bar placed at the top of the wall and at mid-height is typically sufficient.

EXAMPLE 4.7 *Reinforced Concrete Interaction Diagram*

Given Determine interaction diagram for the 8-inch-thick concrete foundation wall in Example 4.5

Wall height = 10 ft
Wall thickness = 8 in
f'_c = 3,000 psi
f_y = 60,000 psi
One No. 5 bar at 24 inches on center ($A_s = 0.155\ in^2/lf$)

Solution

1.
$C_s = A_s f_y$
$= (0.155\ in^2/lf)(60{,}000\ psi) = 9{,}300\ plf$
$C_c = 0.85\ f'_c\ (A_g - A_s)$
$= 0.85\ (3{,}000\ psi)((8\ in)(12\ in/lf) - 0.155\ in^2/lf) = 244{,}405\ plf$
$\phi M_n = 0$
$\phi P_n = \phi\ (C_c + C_s)$
$= 0.7\ (9{,}300\ plf + 244{,}405\ plf) = 177{,}594\ plf \quad (0, 178)$
$\phi P_{n,max} = 0.8\phi P_n$
$= 0.8\ (177{,}594\ plf) = 142{,}080\ plf \quad (0, 142)$

2.
$c = d = 5\ in$
$a = \beta c = 0.85\ (5\ in) = 4.25\ in$
$C_c = 0.85\ abf'_c = 0.85\ (4.25\ in)\ (12\ in)(3{,}000\ psi) = 130{,}050\ plf$
$\phi M_n = \phi C_c\ (d - 0.5a) = 0.7\ (130{,}050\ plf)(5 - 0.5(4.25\ in) = 261{,}725\ in\text{-}lb/lf = 21.8\ ft\text{-}kip/lf$
$\phi P_n = \phi Cc = 0.7\ (130{,}050\ plf) = 91{,}035\ plf \quad (21.8, 91)$

3.
$\varepsilon_c = 0.003$

$$\varepsilon_y = f_y / E_s = \frac{60{,}000\ psi}{29 \times 10^6\ psi} = 2.07 \times 10^{-3} = 0.002$$

$$c = \left(\frac{\varepsilon_c}{\varepsilon_c + 0.5\varepsilon_y}\right) d = \left(\frac{0.003}{0.003 + 0.5\,(0.002)}\right)(5\ in) = 3.72\ in$$

$a = \beta c = 0.85\ (3.72\ in) = 3.16\ in$
$T_s = A_s\ (0.5\ f_y) = (0.155 in^2)(0.5)(60{,}000\ psi) = 4{,}650\ plf$
$C_c = 0.85\ abf'_c = 0.85\ (3.10\ in)(12\ in)(3{,}000\ psi) = 96{,}696\ plf$

$\phi M_n = \phi C_c\ (d - 0.5a) = 0.7\ (96{,}696\ plf)(5in - 0.5(3.16in)) = 231{,}490\ in\text{-}lb/lf = 19.3\ ft\text{-}kip/lf$
$\phi P_n = \phi\ (C_c - T_s) = 0.7\ (96{,}696\ plf - 4{,}650\ plf) = 64{,}432\ plf \quad (19.3, 64)$

4.
$\varepsilon_c = 0.003$

$$\varepsilon_y = f_y / E_s = \frac{60{,}000\ psi}{29 \times 10^6\ psi} = 2.07 \times 10^{-3}$$

$$c = \left(\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_y}\right) d = \left(\frac{0.003}{0.003 + 2.07 \times 10^3}\right)(5\ in) = 2.96\ in$$

$a = \beta c = 0.85\ (2.96\ in) = 2.5\ in$
$C_c = 0.85\ abf'_c = 0.85\ (2.5\ in)(12\ in)(3{,}000\ psi) = 76{,}500\ plf$
$T_s = A_s f_y = (0.155 in^2)(40{,}000\ psi) = 9{,}300\ plf$
$\phi M_n = \phi C_c\ (d - 0.5a) = 0.7\ (76{,}500\ plf)(5\ in - 0.5(2.5\ in)) = 200{,}810\ in\text{-}lb/lf =$ $16.7\ ft\text{-}kip/lf$
$\phi P_n = \phi\ (C_c - T_s) = 0.7\ (76{,}500\ plf - 9{,}300\ plf) = 47{,}040\ plf \quad (16.7, 47)$

5.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.155\,in^2)(60{,}000\,psi)}{0.85(3{,}000\,psi)(12\,in)} = 0.304\,in$$

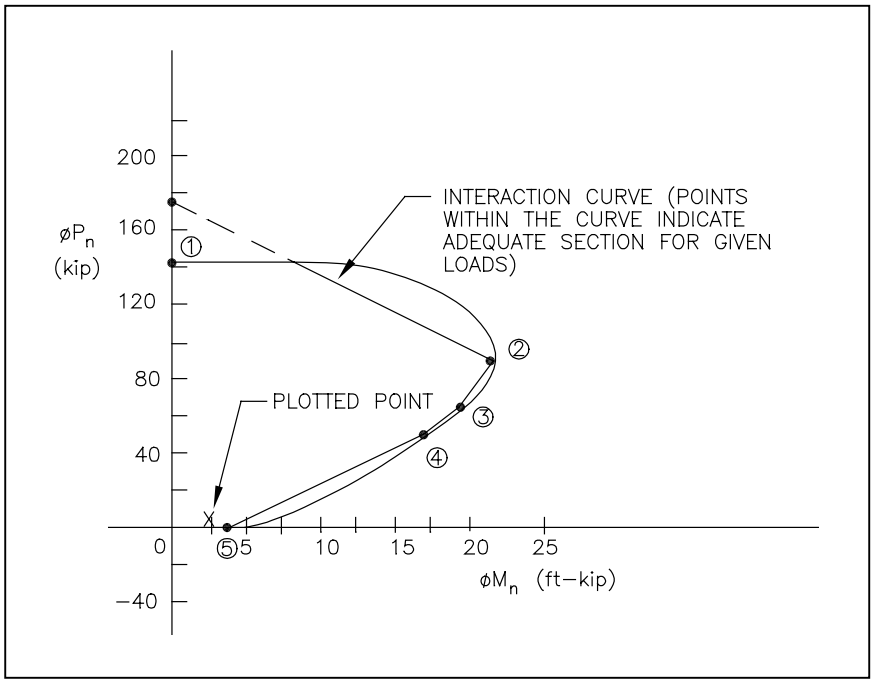
$\phi M_n = \phi A_s f_y (d - 0.5a)$
$= 0.9\,(0.155\,in^2)(60{,}000\,psi)(5\,in - 0.5(0.304\,in)) = 40{,}578\,in\text{-}lb/lf = 3.4\,ft\text{-}kip/lf$
$\phi P_n = 0$ (3.4, 0)

6. Plot the previously calculated points on a graph to determine the interaction diagram boundary for one No. 5 bar at 24 inches on-center vertically in the given wall

PT 1: (0,142)
PT 2: (21.8,91)
PT 3: (19.3,64)
PT 4: (16.7,47)
PT 5: (3.4,0)
PT X: (2.5,2.5)

Conclusion

The point in question lies within the interaction diagram and the references axes; therefore, one No. 5 bar at 24 inches on-center vertically is adequate for the given loading conditions and wall geometry.



Interaction Diagram

EXAMPLE 4.8 Concrete Lintel

Given

f'_c = 3,000 psi
f_y = 60,000 psi
Dead load = 250 plf
Live load = 735 plf
Span = 6.5 ft
Lintel width = 8 in
Lintel depth = 12 in

Find Minimum reinforcement required

Solution

1. Determine reinforcement required for flexure

$\phi M_n \geq M_u$

$$M_u = \frac{wl^2}{12} = \frac{1.2(250\,\text{plf}) + 1.6(735\,\text{plf})}{12}(6.5\,\text{ft})^2 = 5{,}197\ \text{ft-lb}$$

$$\phi M_n = \phi A_s f_y (d - 0.5a)$$

d = 12-in depth - 1.5-in cover - 0.375-in stirrup = 10.125 in

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

set $M_u = \phi M_n$ to solve for A_s

$$M_u = \phi A_s f_y \left(d - \frac{1}{2}\left(\frac{A_s f_y}{0.85 f'_c\, b} \right) \right)$$

$$62{,}364\ \text{in-lb} = (0.9)\, A_s\, (60{,}000\ \text{psi}) \left(10.125\ \text{in} - 0.5\left(\frac{A_s\, 60{,}000\ \text{psi}}{0.85(3{,}000\ \text{psi})(12\ \text{in})} \right) \right)$$

$$0 = 546{,}750 A_s - 52{,}941\, A_s^2 - 62{,}364$$

$A_{s,required} = 0.115\ \text{in}^2$

$\therefore$ Use one No. 4 bar ($A_s = 0.20\ \text{in}^2$)

Check reinforcement ratio

$$\rho = \frac{A_s}{bd} = \frac{0.2\ \text{in}^2}{(10.125\,\text{in})(8\,\text{in})} = 0.0025$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y}\left(\frac{87{,}000}{f_y + 87{,}000} \right) = \frac{0.85(3{,}000\ \text{psi})(0.85)}{60{,}000\ \text{psi}}\left(\frac{87{,}000}{60{,}000\ \text{psi} + 87{,}000} \right) = 0.021$$

$$\rho_{max} = 0.75\rho_b = 0.75(0.021) = 0.016$$

$$\rho_{min} = 0.0012$$

Since $\rho_{max} \geq \rho \geq \rho_{min}$ OK

2. Determine shear reinforcement

$\phi V_n \geq V_u$

$V_u = \frac{wl}{2} = \frac{1.2(250\,\text{plf}) + 1.6(735\,\text{plf})}{2} = (6.5\text{ ft}) = 4{,}797\text{ lb}$

Span-to-depth ratio, $\frac{l}{h} = \frac{(6.5\,\text{ft})(12\,\text{in/ft})}{12\text{in}} = 6.5>5$ $\therefore$ Regular beam

$\phi V_n = \phi V_c + 0 = \phi 2\sqrt{f'_c}\, b_w d = (0.85)(2)\sqrt{3{,}000\,\text{psi}}(8\,\text{in})(10.125\,\text{in}) = 7{,}542\text{ lb}$

$V_u \leq \frac{\phi V_c}{2} = \frac{7{,}542\,\text{lb}}{2} = 3{,}771\text{ lb} < 4{,}797\,\text{lb}$

$\therefore$ Stirrups are required

Since $\phi V_c > V_u > \frac{\phi V_c}{2}$ only the minimum shear reinforcement must be provided.

$A_{v,min} = \frac{50 b_w s}{f_y} = \frac{(50)(8\,\text{in})\frac{(10.125\text{ in})}{2}}{60{,}000\,\text{psi}}$

$=0.034\text{ in}^2$

$\therefore$ Use No. 3 bars

Shear reinforcement is not needed when $\frac{\phi V_c}{2} > V_u$

3,771 lb = 4,797lb - [1.2(250 plf)+1.6(735 plf)]x
x = 0.70ft

Supply No. 3 shear reinforcement spaced 5 in on-center for a distance 0.7 ft from the supports.

3. Check deflection

Find x for transformed area

$h\ x\ \left(x/2\right) = nA_s(d-x)$

$0.5(8\,\text{in})(x)^2 = \left(\frac{29{,}000{,}000\,\text{psi}}{3{,}122{,}019\,\text{psi}}\right)(0.2\,\text{in}^2)(10.125\,\text{in}-x)$

$0 = 4x^2 + 1.86x - 18.8$

x = 1.95 in

Calculate moment of inertia for cracked section and gross section

$I_{CR} = \frac{1}{3}hx^3 + nA_s(d-x)^2$

$= \frac{1}{3}(8\text{ in})(1.95\text{ in})^3 + (9.29)(0.2\text{ in}^2)(10.125\text{ in} - 1.95\,\text{in})^2 = 144\text{ in}^4$

$I_g = \frac{1}{12}bh^3 = \frac{1}{12}(8\,\text{in})(12\,\text{in})^3 = 1{,}152\text{ in}^4$

Calculate modulus of rupture

$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{3{,}000\,\text{psi}} = 411\,\text{psi}$

Calculate cracking moment

$$M_{cr} = \frac{f_r I_g}{Y_t} = \frac{(411\,\text{psi})(1{,}152\ \text{in}^4)}{(0.5)(12\,\text{in})} = 78{,}912\,\text{in}-\text{lb/lf} = 6.6\,\text{kip}-\text{ft/lf}$$

Calculate effective moment of inertia

Since the cracking moment M_{cr} is larger than the actual moment M_u the section is not cracked; thus, $I_e = I_g$.

Calculate deflection

$$\rho_{allow} = \frac{l}{240} = \frac{(6.5\,\text{ft})(12\,\text{in/ft})}{240} = 0.33\ \text{in}$$

$$\rho_{actual} = \frac{5\ wl^4}{384\ E_c I_e}$$

$$\rho_{i(LL)} = \frac{5(735\,\text{plf})(6.5\,\text{ft})^4}{384(3{,}122{,}019\,\text{psi})(1{,}152\,\text{in}^4)(\text{ft}^3/1728\text{in}^3)} = 0.008\,\text{in}$$

$$\rho_{i(DL+20\%LL)} = \frac{5(250\,\text{plf}+(0.20)735\,\text{plf}+(150\,\text{pcf})(0.66\,\text{ft})(1\,\text{ft}))(6.5\,\text{ft})^4}{384(3{,}122{,}019\,\text{psi})(1{,}152\,\text{in}^4)(\text{ft}^3/1{,}728\,\text{in}^3)} = 0.006\,\text{in}$$

$$\Delta_{LT} = \Delta_{i(LL)} + \lambda\Delta_{i(DL+20\%LL)}$$
$$= 0.008\ \text{in} + 2\ (0.0055\ \text{in}) = 0.02\ \text{in}$$

$\rho_{LT} \ll \rho_{allow}$ OK

Conclusion

The minimum reinforcement bar required for an 8-inch x 12-inch concrete lintel spanning 6.5 feet is one No. 4 bar.

EXAMPLE 4.9 *Unreinforced Masonry Wall Design*

Given

Live load = 1,300 plf
Dead load = 900 plf
Weight of wall = 52.5 psf
Moment at top = 0
Masonry weight = 120 pcf
Backfill material = 30 pcf
f'_m = 1,900 psi
Face shell mortar bedding

Assume axial load is in middle one-third of wall

Find Verify if a 10-in-thick unreinforced masonry wall is adequate for the ACI-530 load combination

$$U = D+H$$

Solution

1. Determine loads

Equivalent fluid density of backfill soil (Chapter 3)

$$q_s = K_a w = (0.30)(100 \text{ pcf}) = 30 \text{ pcf}$$

Total lateral earth load

$$R = \tfrac{1}{2} q_s l^2 = \tfrac{1}{2} (30 \text{ pcf})(4 \text{ ft})^2 = 240 \text{ plf}$$
$$x = \tfrac{1}{3} \ell = \tfrac{1}{3} (4 \text{ ft}) = 1.33 \text{ ft}$$

Maximum shear occurs at bottom of wall

$$\Sigma M_{top} = 0$$
$$V_{bottom} = \frac{ql^2}{2} - \frac{ql^3}{6L} = \frac{30 \text{ pcf } (4 \text{ ft})^2}{2} - \frac{30 \text{ pcf } (4 \text{ ft})^3}{6 (8 \text{ ft})} = 200 \text{ plf}$$

Maximum moment and its location

$$x_m = \frac{ql - \sqrt{q^2 l^2 - 2qV_{bottom}}}{q}$$

$$x_m = \frac{30 \text{ pcf } (4 \text{ ft}) - \sqrt{(30 \text{ pcf})^2 (4 \text{ ft})^2 - 2 (30 \text{ pcf}) (200 \text{ plf})}}{(30 \text{ pcf})}$$
$$= 2.37 \text{ ft from base of wall}$$

$$M_{max} = -\frac{qlx_m}{2} + \frac{qx_m^3}{6} + V_{bottom} (x_m)$$
$$= -\frac{30 \text{ pcf } (4 \text{ ft})(2.37 \text{ ft})^2}{2} + \frac{(30 \text{ pcf}) (2.37 \text{ ft})^3}{6} + 200 \text{ plf } (2.37 \text{ ft})$$
$$= 204 \text{ ft-lb/lf}$$

2. Check perpendicular shear

$$\frac{M}{Vd} = \frac{204\,\text{ft}-\text{lb/lf}\ (12\,\text{in/ft})}{200\,\text{plf}\ (9.625\,\text{in})} = 1.27>1$$

$$F_v = \begin{cases} 1.5\sqrt{f'_m} = 1.5\sqrt{1{,}900\ \text{psi}} = 65.4\ \text{psi} \\ 120\ \text{psi} \\ 37\ \text{psi} + 0.45\dfrac{N_v}{A_n} = 37\ \text{psi} + 0.45\dfrac{(900\ \text{plf} + 52.5\ \text{psf}\,(8\,\text{ft} - 2.37\,\text{ft}))}{33\,\text{in}^2} = 53.3\ \text{psi} \end{cases}$$

$$F_v = 53.3\ \text{psi}$$

$$f_v = \frac{3}{2}\left(\frac{V}{A_n}\right) = 1.5\left(\frac{200\,\text{plf}}{(2\,\text{face shells})(1.375\,\text{in})(12\,\text{in})}\right) = 9.1\,\text{psi}$$

The shear is assumed to be resisted by 2 face shells since the wall is unreinforced and uncracked.

$f_v < F_v$ OK

3. Check axial compression

$$A_n = \ell(2b) = (12\ \text{in})(2)(1.375\ \text{in}) = 33\ \text{in}^2$$

$$I = \frac{1}{12}bh^3 + Ad^2$$

$$= 2\left[\frac{1}{12}(12\,\text{in})(1.375\,\text{in})^3 + (12\,\text{in})(1.375\,\text{in})\left(\frac{9.625\,\text{in}}{2} - \frac{1.375\,\text{in}}{2}\right)^2\right]$$

$$= 567\ \text{in}^4$$

$$r = \sqrt{I/A_n} = \sqrt{\frac{567\ \text{in}^4}{33\ \text{in}^2}} = 4.14\ \text{in}$$

$$S = \frac{I}{c} = \frac{567\ \text{in}^4}{\frac{1}{2}(9.625\ \text{in})} = 118\ \text{in}^3$$

$$\frac{h}{r} = \frac{8\,\text{ft}(12\,\text{in/ft})}{4.14\,\text{in}} = 23.2 < 99$$

$$F_a = (0.25\ f'_m)\left[1 - \left(\frac{h}{140r}\right)^2\right] = (0.25)(1{,}900\ \text{psi})\left[1 - \left(\frac{8\ \text{ft}\ (12\ \text{in/ft})}{140\ (4.14\,\text{in})}\right)^2\right] =$$

$$= 462\ \text{psi}$$

$P_{max} = F_aA_n = (462\ \text{psi})(33\ \text{in}^2) = 15{,}246\ \text{plf}$

$P = 900\ \text{plf}$ (given for U=D+H)

$900\ \text{plf} < 15{,}246\ \text{plf}$ OK

Check Euler buckling load

E_m = $900f'_m = 900\ (1{,}900\ psi) = 1.71 \times 10^6$ psi

$e_k = \dfrac{S}{A_n} = \dfrac{118\,in^3}{33\,in^2} = 3.57\,in$ (kern eccentricity)

$$P_e = \frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3$$

$$= \frac{\pi^2\left(1.71x10^6\ psi\right)\left(567\,in^4\right)}{(8\,ft)^2\,(12\,in/ft)^2}\left(1 - 0.577\left(\frac{3.57\,in}{4.14\,in}\right)\right)^3$$

$$= 131{,}703\ plf$$

$P \leq 0.25P_e$ OK

Euler buckling loads are calculated by using actual eccentricities from gravity loads without including effects of lateral loads.

4. Check combined axial compression and flexural capacity

$M = 204$ ft-lb/lf

$P = 900$ plf

virtual eccentricity $e = \dfrac{M}{P} = \dfrac{204\,ft-lb/lf\ (12in/ft)}{900plf} = 2.72$ in

kern eccentricity $e_k = \dfrac{S}{A_n} = \dfrac{118in^3}{33in^2} = 3.57$ in ! GOVERNS

$e < e_k$ ∴ Assume section is uncracked

$$P_e = \frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3$$

$$= \frac{\pi^2\,(900\,plf)(1{,}900\,psi)(567\,in^4)}{(8\,ft\,(12\,in/ft))^2}\left(1 - 0.577\left(\frac{3.57}{4.14}\right)\right)^3$$

$P_e = 131{,}703$ plf

$P < 0.25\ (131{,}703\ plf) = 32{,}926$ plf OK

$$f_a = \frac{P}{A_n} = \frac{900\ plf}{33\ in^2} = 27\ psi$$

$$f_b = \frac{M}{S} = \frac{(900\ plf)(3.57\ in)\left(\frac{2.37\ ft}{8\ ft}\right) + (204\ ft-lb/lf)(12\ in/ft)}{118\ in^3}$$

$$= 29\ psi$$

$F_a = 462$ psi for $h/r \leq 99$

$F_b = 0.33\ f'_m = 0.33\ (1{,}900\ psi) = 627$ psi

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{27\,psi}{462\,psi} + \frac{29\,psi}{627\,psi} = 0.10 \leq 1 \quad \text{OK}$$

5. Check tension capacity from Table 2.2.3.2 for normal to bed joints, hollow, ungrouted (Type M or S mortar)

$$F_t \leq 25 \text{ psi}$$

$$f_t = -\frac{P}{A_n} + \frac{M}{S} = -\frac{900 \text{ plf}}{33 \text{ in}^2} + \frac{3{,}400 \text{ ft} - \text{lb/lf}}{118 \text{ in}^3} = 1.54 \text{ psi}$$

$$f_t < F_t \quad \text{OK}$$

6. Minimum reinforcement

Horizontal reinforcement at 24 inches on-center vertically.

Conclusion

An unreinforced masonry wall is adequate for the ACI-530 load combination evaluated; however, horizontal reinforcement at 24 inches on-center may be optionally provided to control potential shrinkage cracking, particularly in long walls (i.e., greater than 20 to 30 feet long).

If openings are present, use lintels and reinforcement as suggested in Sections 4.5.2.3 and 4.5.2.4.

Note that the calculations have already been completed and that the maximum backfill height calculated for an 8-inch-thick unreinforced masonry wall using hollow concrete masonry is about 5 feet with a safety factor of 4.

EXAMPLE 4.10 Reinforced Masonry Foundation Wall Design

Given

Live load = 1,300 plf
Dead load = 900 plf
Moment at top = 0
Masonry weight = 120 pcf
Wall weight = 52.5 psf
Backfill material = 45 pcf
f'_m = 2,000 psi
Face shell mortar bedding
Type M or S mortar
Wall is partially grouted, one core is grouted at 24 inches on-center
Assume axial load is in middle one-third of wall

Find Verify if one vertical No. 5 bar at 24 inches on-center is adequate for a reinforced concrete masonry foundation wall that is 8 feet high with 7 feet of unbalanced backfill for the ACI-530 load combination

U=D+H

Solution

1. Determine loads

Equivalent fluid density of backfill soil (refer to Chapter 3)

$$q = K_a W = (0.45)(100) = 45 \text{ pcf}$$

Total lateral earth load

$$R = \tfrac{1}{2} ql^2 = \tfrac{1}{2}(45 \text{ pcf})(7 \text{ ft})^2 = 1{,}103 \text{ lb}$$
$$X = \tfrac{1}{3}\ell = \tfrac{1}{3}(7 \text{ ft}) = 2.33 \text{ ft}$$

Maximum shear occurs at bottom of wall

$$\Sigma M_{top} = 0$$
$$V_{bottom} = \frac{ql^2}{2} - \frac{ql^3}{6L} = \frac{45\text{pcf}\,(7\text{ft})^2}{2} - \frac{(45\text{pcf})(7\text{ft})^3}{6(8\text{ft})}$$
$$= 781 \text{ plf}$$

Maximum moment and its location

$$x_m = \frac{ql - \sqrt{q^2l^2 - 2qV_{bottom}}}{q}$$
$$= \frac{(45\text{pcf})(7\text{ft}) - \sqrt{(45\text{pcf})^2(7\text{ft})^2 - 2(45\text{pcf})(781\text{plf})}}{45\text{pcf}}$$
$$= 3.2 \text{ ft from base of wall}$$

$$M_{max} = \frac{qlx_m^2}{2} + \frac{qx_m^3}{6} + V_{bottom}(x_m)$$

$$= \frac{-45\,pcf\,(7\,ft)(3.2\,ft)^2}{2} + \frac{(45\,pcf)(3.2\,ft)^3}{6} + (781\,plf)(3.2\,ft)$$

$$= 1{,}132\ ft\text{-}lb/lf$$

2. Check perpendicular shear

$$\frac{M}{Vd} = \frac{1{,}132\,ft-lb/lf\,(12\,in/ft)}{(781\,plf)(9.625\,in)} = 1.8 > 1$$

$$F_v = 1\sqrt{f'_m} \le 50\ psi$$

$$= 1\sqrt{2{,}000psi} = 44.7psi < 50\ psi$$

F_v = (44.7 psi)(2-ft grouted core spacing) = 89 psi

$A_e = A_{CMU\ faceshells} + A_{core}$

= (24 in-8.375 in)(2)(1.375 in) + (1.125 in + 1.375 in + 5.875 in)(9.625 in)

= 124 in^2

$$f_v = \frac{V}{bd} = \frac{V}{A_e} = \frac{(781\,plf)(2ft\,rebar\,spacing)}{(124\,in^2)} = 13\ psi$$

$f_v < F_v$ OK

This assumes that both mortared face shells are in compression.

3. Check parallel shear

Foundation walls are constrained against lateral loads by the passive pressure of the soil and soil-wall friction. Parallel shear on the foundation wall can be neglected by design inspection.

4. Check axial compression

A_e = 124 in^2

$$I = \frac{1}{12}bh^3 + Ad^2$$

$$= \frac{1}{12}(8.375\,in)(9.625\,in - 2(1.375\,in))$$

$$+ 2\left[\left(\frac{1}{12}\right)(24\,in)(1.375\,in)^3 + (24\,in)(1.375\,in)\left(\frac{9.625\,in}{2} - \frac{1.375\,in}{2}\right)^2\right]$$

= 1,138 in^4

$$r = \sqrt{\frac{I}{A_e}} = \sqrt{\frac{1{,}138\ in^4}{124\ in}} = 3.03\ in$$

$$\frac{h}{r} = \frac{8\,ft\,(12\,in/ft)}{3.03\,in} = 32 < 99$$

$$\therefore F_a = (0.25\,f'_m)\left(1 - \left(\frac{h}{140r}\right)^2\right)$$

$$= 0.25\,(2{,}000\ psi)\left(1 - \left(\frac{(8\,ft)(12\,in/ft)}{140(3.03\,in)}\right)^2\right) = 474\ psi$$

$P_{max} = F_a A_e$ = (474 psi)(124 in^2) = 58,776 lb

P = 900 lb

$P < P_{max}$ OK

5. Check combined axial compression and flexural capacity

M = 1,132 ft-lb/lf
P = 900 plf

virtual eccentricity = e = M/P

$$= \frac{1{,}132\ \text{ft} - \text{lb/lf}\,(12\ \text{in/ft})}{900\ \text{plf}} = 15\ \text{in} \quad \text{!Governs}$$

kern eccentricity = $e_k = \frac{S}{A_e}$

$$= \frac{1{,}138\ \text{in}^4 / 0.5(9.625\ \text{in})}{124\ \text{in}^2} = 1.9\ \text{in}$$

$e > e_k$ ∴ Tension on section, assume cracked

$$f_a = P/A_e = \frac{900\ \text{plf}\,(2\text{ft})}{124\ \text{in}^2} = 14.5\ \text{psi}$$

$$f_b = \frac{M}{S} = \frac{1{,}132\ \text{ft} - \text{lb/lf}\,(12\ \text{in/ft})}{236.5\ \text{in}^3} = 57\ \text{psi}$$

$f_b > f_a$

∴Assume section is cracked

$$F_a = 0.25\ f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right]$$

$$= 0.25\ (2{,}000\ \text{psi}) \left[1 - \left(8\ \text{ft}(12\ \text{in/ft}) / 140\,(3.03\ \text{in})\right)^2\right]$$

$$= 474\ \text{psi}$$

$F_b = 0.33\ f'_m = 0.33\ (2{,}000\ \text{psi}) = 660\ \text{psi}$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$

$$\frac{14.5\ \text{psi}}{474\ \text{psi}} + \frac{57\ \text{psi}}{660\ \text{psi}} = 0.12 \le 1 \quad \text{OK}$$

6. Minimum steel requirement

$$A_{s,req'd} = \frac{M}{F_s d}$$

$$= \frac{(1{,}132\ \text{ft} - \text{lb/lf})\,(12\ \text{in/ft})}{(24{,}000\ \text{psi})\,(0.5)\,(9.625\ \text{in})}$$

$$= 0.12\ \text{in}^2/\text{lf}$$

Minimum vertical reinforcement

$A_{smin} = 0.0013\ bt$

$= (0.0013\ \text{in}^2/\text{lf})(12\ \text{in})(9.625\ \text{in}) = 0.15\ \text{in}^2/\text{lf}$! Governs

No. 5 at 24 inches on-center ($A_s = 0.3\ \text{in}^2(12\ \text{in}/24\ \text{in}) = 0.155\ \text{in}^2$)

$A_{s,actual} > A_{s,required}$ OK

Minimum horizontal reinforcement

$A_{v,hor}$ = 0.0007 bt
= 0.0007 (12 in)(9.625 in) = 0.081 in^2/lf

Use truss-type reinforcement at 24 inches on-center or one No. 5 bar at 48 inches on center (A_s = 0.08 in^2/lf)

7. Check tension

M_t = $A_s dF_s$
= (0.155 in^2)(0.5)(9.625 in)(24,000 psi)
= 17,903 in-lb/lf
M = (1,132 ft-lb/lf)(12 in/ft)
= 13,584 in-lb/lf

$M < M_t$ OK

Conclusion

One vertical No. 5 bar at 24 inches on-center is adequate for the given loading combination. In addition, horizontal truss type reinforcement is recommended at 24 inches (i.e., every third course of block).

Load combination D+H controls design. Therefore, a check of D+L+H is not shown.

Table 4.5 would allow a 10-inch-thick solid unit masonry wall without rebar in soil with 30 pcf equivalent fluid density. This practice has succeeded in residential construction except as reported in places with "heavy" clay soils. Therefore, a design as shown in this example may be replaced by a design in accordance with the applicable residential codes' prescriptive requirements. The reasons for the apparent inconsistency may be attributed to a conservative soil pressure assumption or a conservative safety factor in ACI-530 relative to typical residential conditions.

4.10 References

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CHAPTER 5

Design of Wood Framing

5.1 General

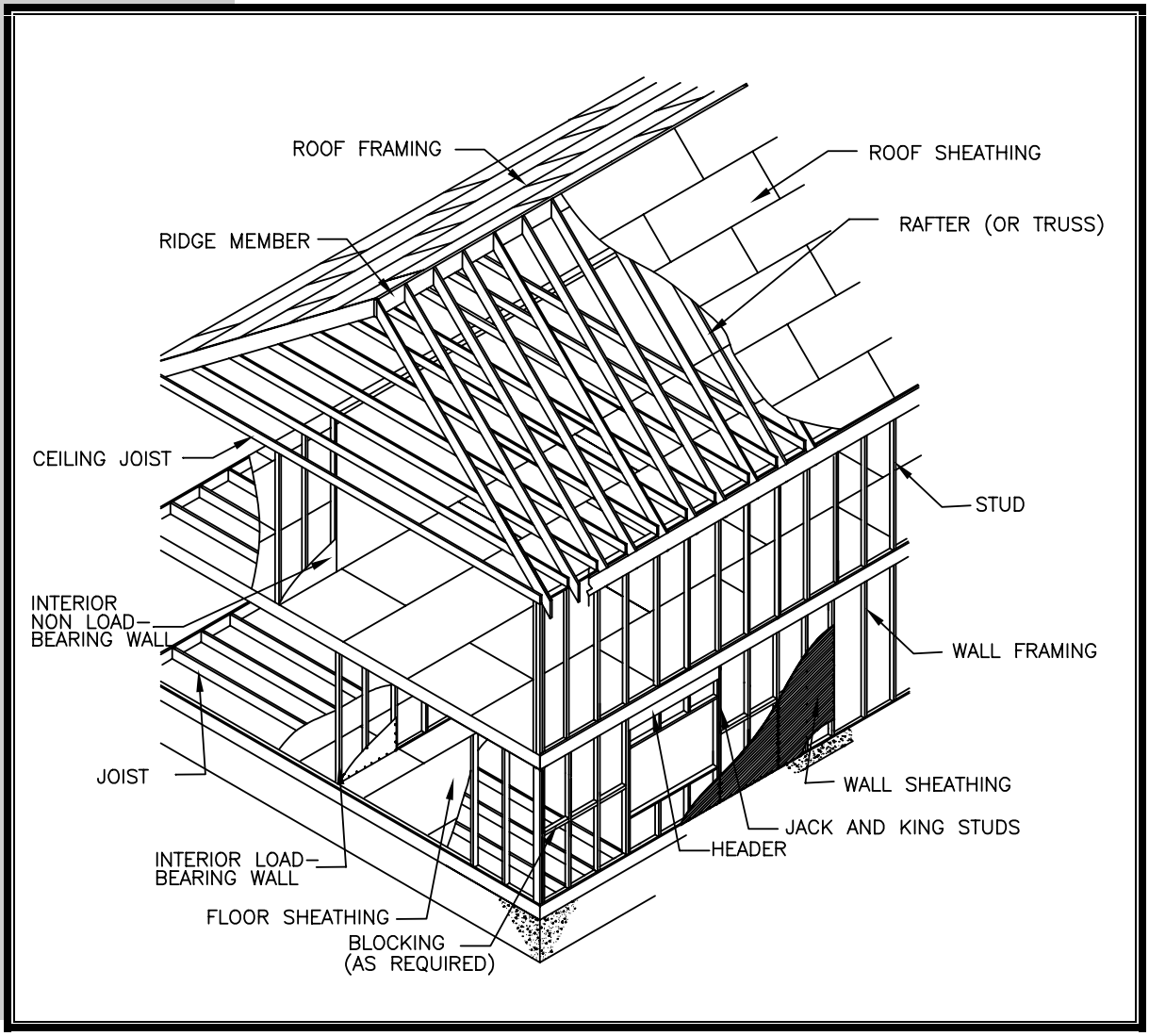
This chapter addresses elements of above-grade structural systems in residential construction. As discussed in Chapter 1, the residential construction material most commonly used above grade in the United States is light-frame wood; therefore, this chapter focuses on structural design that specifies standard dimension lumber and structural wood panels (i.e., plywood and oriented strand board sheathing). Design of the lateral force resisting system (i.e., shearwalls and diaphragms) must be approached from a system design perspective and is addressed in Chapter 6. Connections are addressed in Chapter 7, and their importance relative to the overall performance of wood-framed construction cannot be overemphasized. The basic components and assemblies of a conventional wood frame home are shown in Figure 5.1; the reader is referred to Chapter 1 for more detailed references to house framing and related construction details.

Many elements of a home work together as a system to resist lateral and axial forces imposed on the above-grade structure and transfer them to the foundation. The above-grade structure also helps resist lateral soil loads on foundation walls through connection of floor systems to foundations. Therefore, the issue of system performance is most pronounced in the above-grade assemblies of light-frame homes. Within the context of simple engineering approaches that are familiar to designers, system-based design principles are addressed in this Chapter.

The design of the above-grade structure involves the following structural systems and assemblies:

- floors;
- walls; and
- roofs.

FIGURE 5.1 Components and Assemblies of a Conventional Wood-Framed Home



Each system can be complex to design as a whole; therefore, simple analysis usually focuses on the individual elements that constitute the system. In some cases, "system effects" may be considered in simplified form and applied to the design of certain elements that constitute specifically defined systems. Structural elements that make up a residential structural system include:

- bending members;
- columns;
- combined bending and axial loaded members;
- sheathing (i.e., diaphragm); and
- connections.

The principal method of design for wood-framed construction has historically been allowable stress design (ASD). This chapter uses the most current version of the ASD method (AF&PA, 1997), although the load resistance factored design method (LRFD) is now available as an alternative (AF&PA, 1996a). The ASD method is detailed in the *National Design Specification for Wood Construction* (NDS) and its supplement (NDS-S). The designer is encouraged to obtain the NDS commentary to develop a better understanding of the rationale and substantiation for the NDS (AF&PA, 1999).

This chapter looks at the NDS equations in general and includes design examples that detail the appropriate use of the equations for specific structural elements or systems in light, wood-framed construction. The discussion focuses primarily on framing with traditional dimension lumber but gives some consideration to common engineered wood products. Other wood framing methods, such as post-and-beam construction, are not explicitly addressed in this chapter, although much of the information is relevant. However, system considerations and system factors presented in this chapter are only relevant to light, wood-framed construction using dimension lumber.

Regardless of the type of structural element to analyze, the designer must first determine nominal design loads. The loads acting on a framing member or system are usually calculated in accordance with the applicable provisions of the locally approved building code and engineering standards. The nominal design loads and load combinations used in this chapter follow the recommendations in Chapter 3 for residential design.

While prescriptive design tables (i.e., span tables) and similar design aids commonly used in residential applications are not included herein, the designer may save considerable effort by consulting such resources. Most local, state, or national model building codes such as the *One- and Two-Family Dwelling Code* (ICC, 1998) contain prescriptive design and construction provisions for conventional residential construction. Similar prescriptive design aids and efficient framing practices can be found in *Cost-Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994). For high wind conditions, prescriptive guidelines for design and construction may be found in the *Wood Frame Construction Manual for One- and Two-Family Dwellings* (AFPA, 1996b). The designer is also encouraged to obtain design data on a variety of proprietary engineered wood products that are suitable for many special design needs in residential construction. However, these materials generally should not be viewed as simple “one-to-one” substitutes for conventional wood framing and any special design and construction requirements should be carefully considered in accordance with the manufacturer’s recommendation or applicable code evaluation reports.

5.2 Material Properties

It is essential that a residential designer specifying wood materials appreciate the natural characteristics of wood and their effect on the engineering properties of lumber. A brief discussion of the properties of lumber and structural wood panels follows.

5.2.1 Lumber

General

As with all materials, the designer must consider wood's strengths and weaknesses. A comprehensive source of technical information on wood characteristics is the *Wood Engineering Handbook, Second Edition* (Forest Products Laboratory, 1990). For the most part, the knowledge embodied in the handbook is reflected in the provisions of the NDS and the NDS Supplement (NDS-S) design data; however, many aspects of wood design require good judgment.

Wood is a natural material that, as a structural material, demonstrates unique and complex characteristics. Wood's structural properties can be traced back to the material's natural composition. Foremost, wood is a nonhomogeneous, non-isotropic material, and thus exhibits different structural properties depending on the orientation of stresses relative to the grain of the wood. The grain is produced by a tree's annual growth rings, which determine the properties of wood along three orientations: tangential, radial, and longitudinal.

Given that lumber is cut from logs in the longitudinal direction, the grain is parallel to the length of a lumber member. Depending on where the lumber is cut relative to the center of a log (i.e., tangential versus radial), properties vary across the width and thickness of an individual member.

Wood Species

Structural lumber can be manufactured from a variety of wood species; however, the various species used in a given locality are a function of the economy, regional availability, and required strength properties. A wood species is classified as either hardwood or softwood. *Hardwoods* are broad-leafed deciduous trees while s*oftwoods* (i.e., conifers) are trees with needle-like leaves and are generally evergreen.

Most structural lumber is manufactured from softwoods because of the trees' faster growth rate, availability, and workability (i.e., ease of cutting, nailing, etc.). A wood species is further classified into groups or combinations as defined in the NDS. Species within a group have similar properties and are subject to the same grading rules. Douglas Fir-Larch, Southern Yellow Pine, Hem-Fir, and Spruce-Pine-Fir are species groups that are widely used in residential applications in the United States.

Lumber Sizes

Wood members are referred to by nominal sizes (e.g., 2x4); however, true dimensions are somewhat less. The difference occurs during the dressing stage of the lumber process, when each surface of the member is planed to its final dressed dimension after shrinkage has occurred as a result of the drying or "seasoning" process. Generally, there is a 1/4- to 3/4-inch difference between the nominal and dressed sizes of "dry" sawn lumber (refer to NDS-S Table 1B for specific dimensions). For example, a 2x4 is actually 1.5 inches by 3.5 inches, a 2x10 is 1.5

inches by 9.25 inches, and a 1x4 is 3/4-inch by 3.5 inches. This guide uses nominal member size, but it is important to note that the designer must apply the actual dimensions of the lumber when analyzing structural performance or detailing construction dimensions.

Based on the expected application, the tabulated values in the NDS are classified by the species of wood as well as by the nominal size of a member. Typical NDS classifications follow:

- *Boards* are less than 2 inches thick.
- *Dimension lumber* is a minimum of 2 inches wide and 2 to 4 inches thick.
- *Beams and stringers* are a minimum of 5 inches thick, with the width at least 2 inches greater than the thickness dimension.
- *Posts and timbers* are a minimum of 5 inches thick, and the width does not exceed the thickness by more than 2 inches.
- *Decking* is 2 to 4 inches thick and loaded in the weak axis of bending for a roof, floor, or wall surface.

Most wood used in light-frame residential construction takes the form of dimension lumber.

Lumber Grades

Lumber is graded in accordance with standardized grading rules that consider the effect of natural growth characteristics and "defects," such as knots and angle of grain, on the member's structural properties. Growth characteristics reduce the overall strength of the member relative to a "perfect," clear-grained member without any natural defects. Most lumber is visually graded, although it can also be machine stress-rated or machine evaluated.

Visually graded lumber is graded by an individual who examines the wood member at the mill in accordance with an approved agency's grading rules. The grader separates wood members into the appropriate grade classes. Typical visual grading classes in order of decreasing strength properties are Select Structural, No. 1, No. 2, Stud, etc. Refer to the NDS Supplement (NDS-S) for more information on grades of different species of lumber. The designer should consult a lumber supplier or contractor regarding locally available lumber species and grades.

Machine stress rated (MSR) and *machine evaluated lumber* (MEL) is subjected to nondestructive testing of each piece. The wood member is then marked with the appropriate grade stamp, which includes the allowable bending stress (F_b) and the modulus of elasticity (E). This grading method yields lumber with more consistent structural properties than visual grading only.

While grading rules vary among grading agencies, the U.S. Department of Commerce has set forth minimums for voluntary adoption by the recognized

lumber grading agencies. For more information regarding grading rules, refer to *American Softwood Lumber Voluntary Product Standard* (USDOC PS-20), which is maintained by the National Institute for Standards and Technology (NIST, 1994). NDS-S lists approved grading agencies and roles.

Moisture Content

Wood properties and dimensions change with moisture content (MC). Living wood contains a considerable amount of free and bound water. Free water is contained between the wood cells and is the first water to be driven off in the drying process. Its loss affects neither volume nor structural properties. Bound water is contained within the wood cells and accounts for most of the moisture under 30 percent; its loss results in changes in both volume (i.e., shrinkage) and structural properties. The strength of wood peaks at about 10 to 15 percent MC.

Given that wood generally has an MC of more than 30 percent when cut and may dry to an equilibrium moisture content (EMC) of 8 to 10 percent in protected environment, it should be sufficiently dried or seasoned before installation. Proper drying and storage of lumber minimizes problems associated with lumber shrinkage and warping. A minimum recommendation calls for using "surface dry" lumber with a maximum 19 percent MC. In uses where shrinkage is critical, specifications may call for "KD-15," which is kiln-dried lumber with a maximum moisture content of 15 percent. The tabulated design values in the NDS are based on a moisture content of 19 percent for dimension lumber.

The designer should plan for the vertical movement that may occur in a structure as a result of shrinkage. For more complicated structural details that call for various types of materials and systems, the designer might have to account for differential shrinkage by isolating members that will shrink from those that will maintain dimensional stability. The designer should also detail the structure such that shrinkage is as uniform as possible, thereby minimizing shrinkage effects on finish surfaces. When practical, details that minimize the amount of wood transferring loads perpendicular-to-grain are preferable.

Shrink and swell can be estimated in accordance with Section 5.3.2 for the width and thickness of wood members (i.e., tangentially and radially with respect to annual rings). Shrinkage in the longitudinal direction of a wood member (i.e., parallel to grain) is negligible.

Durability

Moisture is a primary factor affecting the durability of lumber. Fungi, which feed on wood cells, require moisture, air, and favorable temperatures to survive. When wood is subject to moisture levels above 20 percent and other favorable conditions, decay begins to set in. Therefore, it is important to protect wood materials from moisture, by:

- limiting end use (e.g., specifying interior applications or isolating lumber from ground contact);
- using a weather barrier (e.g., siding, roofing, building wrap, flashing, etc.);
- applying a protective coating (e.g., paint, water repellent, etc.);

- installing roof overhangs and gutters; and
- specifying preservative-treated or naturally decay-resistant wood.

For homes, an exterior weather barrier (e.g., roofing and siding) protects most structural wood. However, improper detailing can lead to moisture intrusion and decay. Problems are commonly associated with improper or missing flashing and undue reliance on caulking to prevent moisture intrusion. For additional information and guidance on improving the durability of wood in buildings, refer to *Prevention and Control of Decay in Homes* (HUD, 1978).

Wood members that are in ground contact should be preservative treated. The most common lumber treatment is CCA (copper-chromium-arsenate), which should be used for applications such as sill plates located near the ground or for exterior decks. It is important to specify the correct level of treatment (0.4 pcf retention for nonground-contact exterior exposure and 0.6 pcf for ground contact).

Termites and other wood-destroying insects (e.g., carpenter ants, boring beetles, etc.) attack wood materials. Some practical solutions include: the chemical treatment of soil; the installation of physical barriers (e.g., termite shields); and the specification of treated lumber.

Termites are a special problem in warmer climates, although they also plague many other areas of the United States. The most common termites are "subterranean" termites that nest in the ground and enter wood that is near or in contact with damp soil. They gain access to above-grade wood through cracks in the foundation or through shelter tubes (i.e., mud tunnels) on the surface of foundation walls. Since the presence of termites lends itself to be visual to detection, wood-framed homes require periodic inspection for signs of termites.

5.2.2 Structural Wood Panels

Historically, boards were used for roof, floor, and wall sheathing; in the last 30 years, however, structural wood panel products have come to dominate the sheathing market. Structural wood panel products are more economical and efficient and can be stronger than traditional board sheathing. Structural wood panel products primarily include plywood and oriented strand board (OSB).

Plywood is manufactured from wood veneers glued together under high temperature and pressure. Each veneer or ply is placed with its grain perpendicular to the grain of the previous layer. The outer layers are placed with their grain parallel to the longer dimension of the panel. Thus, plywood is stronger in bending along the long direction and should be placed with the long dimension spanning floor and roof framing members. The number of plies typically ranges from 3 to 5. Oriented strand board is manufactured from thin wood strands glued together under high temperature and pressure. The strands are layered and oriented to produce strength properties similar to plywood; therefore, the material is used for the same applications as plywood.

The designer should specify the grade and span rating of structural wood panels to meet the required application and loading condition (i.e., roof, wall or floor). The most common panel size is 4x8 feet panels, with thicknesses typically ranging from 3/8-inch to more than 1 inch. Panels can be ordered in longer lengths for special applications.

Plywood is performance-rated according to the provisions of USDOC PS-1 for industrial and construction plywood (NIST, 1995). OSB products are performance-rated according to the provisions of USDOC PS-2 (NIST, 1992). However, these standards are voluntary and not all wood-based panel products are rated accordingly. The APA–Engineered Wood Association's (formerly American Plywood Association) rating system for structural wood panel sheathing products and those used by other structural panel trademarking organizations are based on the U.S. Department of Commerce voluntary product standards.

The veneer grade of plywood is associated with the veneers used on the exposed faces of a panel as follows:

GradeA: The highest-quality veneer grade, which is intended for cabinet or furniture use.
Grade B: A high-quality veneer grade, which is intended for cabinet or furniture use with all defects repaired.
Grade C: The minimum veneer grade, which is intended for exterior use.
Grade D: The lowest-quality veneer grade, which is intended for interior use or where protected from exposure to weather.

The wood strands or veneer layers used in wood structural panels are bonded with adhesives and they vary in moisture resistance. Therefore, wood structural panels are also classified with respect to end-use exposure as follows:

- *Exterior* panels are designed for applications with permanent exposure to the weather or moisture.
- *Exposure 1* panels are designed for applications where temporary exposure to the weather due to construction sequence may be expected.
- *Exposure 2* panels are designed for applications with a potential for high humidity or wetting but are generally protected during construction.
- *Interior* panels are designed for interior applications only.

Typical span ratings for structural wood panels specify either the maximum allowable center-to-center spacing of supports (e.g., 24 inches on center for roof, floor, or wall) or two numbers separated by a slash to designate the allowable center-to-center spacing of roof and floor supports, respectively (e.g., 48/24). Even though the second rating method does not specifically indicate wall stud spacing, the panels may also be used for wall sheathing. The *Design and Construction Guide: Residential and Commercial* provides a correlation between roof/floor ratings and allowable wall support spacing (APA, 1998a). The *Load-Span Tables for APA Structural-Use Panels* (APA, 1999) provided span ratings for various standard and nonstandard loading conditions and deflection limits.

5.2.3 Lumber Design Values

The NDS-S provides tabulated design stress values for bending, tension parallel to grain, shear parallel to grain, compression parallel and perpendicular to

grain, and modulus of elasticity. In particular, the 1997 edition of the NDS includes the most up-to-date design values based on test results from an eight-year full-scale testing program that uses lumber samples from mills across the United States and Canada.

Characteristic structural properties for use in allowable stress design (ASTM D1990) and load and resistance factor design (ASTM D5457) are used to establish design values (ASTM, 1998a; ASTM, 1998b). Test data collected in accordance with the applicable standards determine a characteristic strength value for each grade and species of lumber. The value is usually the mean (average) or fifth percentile test value. The fifth percentile represents the value that 95 percent of the sampled members exceeded. In ASD, characteristic structural values are multiplied by the reduction factors in Table 5.1. The reduction factors are implicit in the allowable values published in the NDS-S for standardized conditions. The reduction factor normalizes the lumber properties to a standard set of conditions related to load duration, moisture content, and other factors. It also includes a safety adjustment if applicable to the particular limit state (i.e., ultimate capacity). Therefore, for specific design conditions that differ from the standard basis, design property values should be adjusted as described in Section 5.2.4.

The reduction factors in Table 5.1 are derived as follows as reported in ASTM D2915 (ASTM, 1997):

- F_b reduction factor = (10/16 load duration factor)(10/13 safety factor);
- F_t reduction factor = (10/16 load duration factor)(10/13 safety factor);
- F_v reduction factor = (10/16 load duration factor)(4/9 stress concentration factor) (8/9 safety factor);
- F_c reduction factor = (2/3 load duration factor)(4/5 safety factor); and
- $F_{c\perp}$ reduction factor = (2/3 end position factor)

5.2.4 Adjustment Factors

The allowable values published in the NDS-S are determined for a standard set of conditions. Yet, given the many variations in the characteristics of wood that affect the material's structural properties, several adjustment factors are available to modify the published values. For efficient design, it is important to use the appropriate adjustments for conditions that vary from those used to derive the standard design values. Table 5.2 presents adjustment factors that apply to different structural properties of wood. The following sections briefly discuss the adjustment factors most commonly used in residential applications. For information on other adjustment factors, refer to the NDS, NDS-S, and the NDS commentary.

TABLE 5.1 *Design Properties and Associated Reduction Factors for ASD*

Stress Property	Reduction Factor	Basis of Estimated Characteristic Value from Test Data	Limit State	ASTM Designation
Extreme fiber stress in bending, F_b	$\frac{1}{2.1}$	Fifth percentile	Ultimate capacity	D1990
Tension parallel to grain, F_t	$\frac{1}{2.1}$	Fifth percentile	Ultimate capacity	D1990
Shear parallel to grain, F_v	$\frac{1}{4.1}$	Fifth percentile	Ultimate capacity	D245
Compression parallel to grain, F_c	$\frac{1}{1.9}$	Fifth percentile	Ultimate capacity	D1990
Compression perpendicular to grain, $F_{c\perp}$	$\frac{1}{1.5}$	Mean	0.04" deflection[1]	D245
Modulus of elasticity, E	$\frac{1}{1.0}$	Mean	Proportional limit[2]	D1990

Sources: ASTM, 1998a; ASTM, 1998c.
Notes:
[1]The characteristic design value for $F_{c\perp}$ is controlled by a deformation limit state. In fact, the lumber will densify and carry an increasing load as it is compressed.
[2]The proportional limit of wood load-deformation behavior is not clearly defined because it is nonlinear. Therefore, designation of a proportional limit is subject to variations in interpretation of test data.

TABLE 5.2 *Adjustment Factor Applicability to Design Values for Wood*

Design Properties[1]	Adjustment Factor[2]														
	C_D	C_r	C_H	C_F	C_P	C_L	C_M	C_{fu}	C_b	C_T	C_V	C_t	C_i	C_c	C_f
F_b	✔	✔		✔		✔	✔	✔			✔	✔	✔	✔	✔
F_t	✔			✔			✔					✔	✔		
F_v	✔		✔				✔					✔	✔		
$F_{c\perp}$							✔		✔			✔	✔		
F_c	✔			✔	✔		✔					✔	✔		
E							✔			✔		✔	✔		

Source: Based on NDS•2.3 (AF&PA, 1997).
Notes:
[1]Basic or unadjusted values for design properties of wood are found in NDS-S. See Table 5.1 for definitions of design properties.
[2]Shaded cells represent factors most commonly used in residential applications; other factors may apply to special conditions.

Key to Adjustment Factors:

- C_D, Load Duration Factor. Applies when loads are other than "normal" 10-year duration (see Section 5.2.4.1 and NDS•2.3.2).

- C_r, Repetitive Member Factor. Applies to bending members in assemblies with multiple members spaced at maximum 24 inches on center (see Section 5.2.4.2 and NDS•4.3.4).

- C_H, Horizontal Shear Factor. Applies to individual or multiple members with regard to horizontal, parallel-to-grain splitting (see Section 5.2.4.3 and NDS-S).

- C_F, Size Factor. Applies to member sizes/grades other than "standard" test specimens, but does not apply to Southern Yellow Pine (see Section 5.2.4.4 and NDS-S).

- C_P, Column Stability Factor. Applies to lateral support condition of compression members (see Section 5.2.4.5 and NDS•3.7.1).

- C_L, Beam Stability Factor. Applies to bending members not subject to continuous lateral support on the compression edge (see Section 5.2.4.6 and NDS•3.3.3).

- C_M, Wet Service Factor. Applies where the moisture content is expected to exceed 19 percent for extended periods (see NDS-S).

- C_{fu}, Flat Use Factor. Applies where dimension lumber 2 to 4 inches thick is subject to a bending load in its weak axis direction (see NDS-S).

- C_b, Bearing Area Factor. Applies to members with bearing less than 6 inches and not nearer than 3 inches from the members' ends (see NDS•2.3.10).

- C_T, Buckling Stiffness Factor. Applies only to maximum 2x4 dimension lumber in the top chord of wood trusses that are subjected to combined flexure and axial compression (see NDS•4.4.3).

- C_V, Volume Factor. Applies to glulam bending members loaded perpendicular to the wide face of the laminations in strong axis bending (see NDS•5.3.2).

- C_t, Temperature Factor. Applies where temperatures exceed 100^oF for long periods; not normally required when wood members are subjected to intermittent higher temperatures such as in roof structures (see NDS•2.4.3 and NDS•Appendix C).

- C_i, Incising Factor. Applies where structural sawn lumber is incised to increase penetration of preservatives with small incisions cut parallel to the grain (see NDS•2.3.11).

- C_c, Curvature Factor. Applies only to curved portions of glued laminated bending members (see NDS•5.3.4).

- C_f, Form Factor. Applies where bending members are either round or square with diagonal loading (see NDS•2.3.8).

5.2.4.1 Load Duration Factor (C_D)

Lumber strength is affected by the cumulative duration of maximum variable loads experienced during the life of the structure. In other words, strength is affected by both the load intensity and its duration (i.e., the load history). Because of its natural composition, wood is better able to resist higher short-term loads (i.e., transient live loads or impact loads) than long-term loads (i.e., dead loads and sustained live loads). Under impact loading, wood can resist about twice as much stress as the standard 10-year load duration (i.e., "normal duration") to which wood bending stress properties are normalized in the NDS.

When other loads with different duration characteristics are considered, it is necessary to modify certain tabulated stresses by a load duration factor (C_D) as shown in Table 5.3. Values of the load duration factor, C_D, for various load types

are based on the total accumulated time effects of a given type of load during the useful life of a structure. C_D increases with decreasing load duration.

Where more than one load type is specified in a design analysis, the load duration factor associated with the shortest duration load is applied to the entire combination of loads. For example, for the load combination, *Dead Load + Snow Load + Wind Load*, the load duration factor, C_D, is equal to 1.6.

TABLE 5.3 Recommended Load Duration Factors for ASD

Load Type	Load Duration	Recommended C_D Value
Permanent (dead load)	Lifetime	0.9
Normal	Ten years	1.0
Occupancy (live load)[1]	Ten years to seven days	1.0 to 1.25
Snow[2]	One month to seven days	1.15 to 1.25
Temporary construction	Seven days	1.25
Wind and seismic[3]	Ten minutes to one minute	1.6 to 1.8
Impact	One second	2.0

Source: Based on NDS•2.3.2 and NDS•Appendix B (AF&PA, 1997).
Notes:
[1]The NDS uses a live load duration of ten years (C_D = 1.0). The factor of 1.25 is consistent with the time effect factor for live load used in the new wood LRFD provisions (AF&AP, 1996a).
[2]The NDS uses a snow load duration of one month (C_D = 1.15). The factor of 1.25 is consistent with the time effect factor for snow load used in the new wood LRFD provisions (AF&PA, 1996a).
[3]The NDS uses a wind and seismic load duration of ten minutes (C_D = 1.6). The factor may be as high as 1.8 for earthquake loads which generally have a duration of less than 1 minute with a much shorter duration for ground motions in the design level range.

5.2.4.2 Repetitive Member Factor (C_r)

When three or more parallel dimension lumber members are spaced a maximum of 24 inches on center and connected with structural sheathing, they comprise a structural "system" with more bending capacity than the sum of the single members acting individually. Therefore, most elements in a house structure benefit from an adjustment for the system strength effects inherent in repetitive members.

The tabulated design values given in the NDS are based on single members; thus, an increase in allowable stress is permitted in order to account for repetitive members. While the NDS recommends a repetitive member factor of 1.15 or a 15 percent increase in bending strength, system assembly tests have demonstrated that the NDS repetitive member factor is conservative for certain conditions. In fact, test results from several studies support the range of repetitive member factors shown in Table 5.4 for certain design applications. As shown in Table 5.2, the adjustment factor applies only to extreme fiber in bending, F_b. Later sections of Chapter 5 cover other system adjustments related to concentrated loads, header framing assemblies, and deflection (stiffness) considerations.

TABLE 5.4 Recommended Repetitive Member Factors for Dimension Lumber Used in Framing Systems[1,2]

Application	Recommended C_r Value	References
Two adjacent members sharing load[3]	1.1 to 1.2	AF&PA,1996b HUD, 1999
Three adjacent members sharing load[3]	1.2 to 1.3	ASAE, 1997
Four or more adjacent members sharing load[3]	1.3 to 1.4	ASAE, 1997
Three or more members spaced not more than 24 inches on center with suitable surfacing to distribute loads to adjacent members (i.e., decking, panels, boards, etc.)[4]	1.15	NDS
Wall framing (studs) of three or more members spaced not more than 24 inches on center with minimum 3/8-inch-thick wood structural panel sheathing on one side and 1/2-inch thick gypsum board on the other side[5]	1.5–2x4 or smaller 1.35–2x6 1.25–2x8 1.2–2x10	AF&PA, 1996b SBCCI, 1999 Polensek, 1975

Notes:
[1]NDS recommends a C_r value of 1.15 only as shown in the table. The other values in the table were obtained from various codes, standards, and research reports as indicated.
[2]Dimension lumber bending members are to be parallel in orientation to each other, continuous (i.e., not spliced), and of the same species, grade, and size. The applicable sizes of dimension lumber range from 2x4 to 2x12.
[3]C_r values are given as a range and are applicable to built-up columns and beams formed of continuous members with the strong-axis of all members oriented identically. In general, a larger value of C_r should be used for dimension lumber materials that have a greater variability in strength (i.e., the more variability in strength of individual members the greater the benefit realized in forming a built-up member relative to the individual member strength). For example, a two-ply built-up member of No. 2 grade (visually graded) dimension lumber may qualify for use of a C_r value of 1.2 whereas a two-ply member of No. 1 dense or mechanically graded lumber may qualify for a C_r value of 1.1. The individual members should be adequately attached to one another or the load introduced to the built-up member such that the individual members act as a unit (i.e., all members deflect equally) in resisting the bending load. For built-up bending members with non-continuous plys (i.e., splices), refer to ASAE EP 559 (ASAE, 1997). For built-up columns subject to weak axis bending load or buckling, refer to ASAE EP 559 and NDS•15.3.
[4]Refer to NDS•4.3.4 and the NDS *Commentary* for additional guidance on the use of the 1.15 repetitive member factor.
[5]The C_r values are based on wood structural panel attachment to wall framing using 8d common nails spaced at 12 inches on center. For fasteners of a smaller diameter, multiply the C_r values by the ratio of the nail diameter to that of an 8d common nail (0.131 inch diameter). The reduction factor applied to C_r need not be less than 0.75 and the resulting value of C_r should not be adjusted to less than 1.15. Doubling the nailing (i.e., decreasing the fastener spacing by one-half) can increase the C_r value by 16 percent (Polensek, 1975).

With the exception of the 1.15 repetitive member factor, the NDS does not currently recognize the values in Table 5.4. Therefore, the values in Table 5.4 are provided for use by the designer as an "alternative" method based on various sources of technical information including certain standards, code recognized guidelines, and research studies. For more information on system effects, consult the following sample of references:

"Structural Performance of Light-Frame Truss-Roof Assemblies" (Wolfe, 1996).

"Performance of Light-Frame Redundant Assemblies" (Wolfe, 1990).

"Reliability of Wood Systems Subjected to Stochastic Live Loads" (Rosowsky and Ellingwood, 1992).

"System Effects in Wood Assemblies" (Douglas and Line, 1996).

Design Requirements and Bending Properties for Mechanically Laminated Columns (EP 559) (ASAE, 1997).

Rational Design Procedure for Wood Stud Walls Under Bending and Compression Loads (Polensek, 1975).

Stress and Deflection Reduction in 2x4 Studs Spaced 24 Inches On Center Due to the Addition of Interior and Exterior Surfacings (NAHBRF, 1974).

Structural Reliability Analysis of Wood Load Sharing Systems (Bonnicksen and Suddarth, 1965).

System Performance of Wood Header Assemblies (HUD, 1999).

Wall & Floor Systems: Design and Performance of Light-Frame Structures (FPRS, 1983).

5.2.4.3 Horizontal Shear Factor (C_H)

Given that lumber does not dry uniformly, it is subject to warping, checking, and splitting, all of which reduce the strength of a member. The horizontal stress values in the NDS-S conservatively account for any checks and splits that may form during the seasoning process and, as in the worst-case values, assume substantial horizontal splits in all wood members. Although a horizontal split may occur in some members, all members in a repetitive member system rarely experience such splits. Therefore, a C_H of greater than 1.0 should typically apply when repetitive framing or built-up members are used. For members with no splits C_H equals 2.0.

In addition, future allowable horizontal shear values will be increased by a factor of 2 or more because of a recent change in the applicable standard regarding assignment of strength properties. The change is a result of removing a conservative adjustment to the test data whereby a 50 percent reduction for checks and splits was applied in addition to a 4/9 stress concentration factor as described in Section 5.2.3. As an interim solution, a shear adjustment factor, C_H, of 2.0 should therefore apply to all designs that use horizontal shear values in 1997 and earlier editions of the NDS. As shown in Table 5.2, the C_H factor applies only to the allowable horizontal shear stress, F_v. As an interim consideration regarding horizontal shear at notches and connections in members, a C_H value of 1.5 is recommended for use with provisions in NDS•3.4.4 and 3.4.5 for dimension lumber only.

5.2.4.4 Size Factor (C_F)

Tabulated design values in the NDS-S are based on testing conducted on members of certain sizes. The specified depth for dimension lumber members subjected to testing is 12 inches for No. 3 or better, 6 inches for stud-grade members, and 4 inches for construction-, standard- or utility-grade members (i.e., C_F=1.0).

The size of a member affects unit strength because of the member's relationship to the likelihood of naturally occurring defects in the material.

Therefore, an adjustment to certain tabulated values is appropriate for sizes other than those tested; however, the tabulated values for Southern Yellow Pine have already been adjusted for size and do not require application of C_F. Table 5.2 indicates the tabulated values that should be adjusted to account for size differences. The adjustment applies when visually graded lumber is 2 to 4 inches thick or when a minimum 5-inch-thick rectangular bending member exceeds 12 inches in depth. Refer to NDS-S for the appropriate size adjustment factor.

5.2.4.5 Column Stability Factor (C_P)

Tabulated compression design values in the NDS-S are based on the assumption that a compression member is continuously supported along its length to prevent lateral displacement in both the weak and strong axes. When a compression member is subject to continuous lateral support in at least two orthogonal directions, Euler buckling cannot occur. However, many compression members (e.g., interior columns or wall framing) do not have continuous lateral support in two directions.

The column stability factor, C_P adjusts the tabulated compression stresses to account for the possibility of column buckling. For rectangular or non-symmetric columns, C_p must be determined for both the weak- and strong-axis bracing conditions. C_p is based on end-fixity, effective length of the member between lateral braces, and the cross-sectional dimensions of the member that affect the slenderness ratio used in calculating the critical buckling stress. Given that the Euler buckling effect is associated only with axial loads, the C_P factor applies to the allowable compressive stress parallel to grain, F_c, as shown in Table 5.2. Refer to the NDS for the equations used to calculate the column stability factor.

5.2.4.6 Beam Stability Factor (C_L)

The tabulated bending design values, F_b, given in the NDS-S are applicable to bending members that are either braced against lateral-torsional buckling (i.e., twisting) or stable without bracing (i.e., depth is no greater than the breadth of the member). Most bending members in residential construction are laterally supported on the compression edge by some type of sheathing product. The beam stability factor does, however, apply to conditions such as ceiling joists supporting unfinished attic space. When a member does not meet the lateral support requirements of NDS•3.3.3 or the stability requirements of NDS•4.4.1, the designer should modify the tabulated bending design values by using the beam stability factor, C_L, to account for the possibility of lateral-torsional buckling. For glued laminated timber bending members, the volume factor (C_V) and beam stability factor (C_L) are not applied simultaneously; thus, the lesser of these factors applies. Refer to the NDS•3.3.3 for the equations used to calculate C_L.

5.3 Structural Evaluation

As with any structural design, the designer should perform several checks with respect to various design factors. This section provides an overview of

checks specified in the NDS and specifies several design concerns that are not addressed by the NDS. In general, the two categories of structural design concerns are:

Structural Safety (strength)

- Bending and lateral stability
- Horizontal Shear
- Bearing
- Combined bending and axial loading
- Compression and column stability
- Tension

Structural Serviceability

- Deflection due to bending
- Floor vibration
- Shrinkage

The remainder of this chapter applies these design checks to examples of different structural systems and elements in a home. In addition, given that the intent of this guide is to provide supplemental instruction for the use of the NDS in the efficient design of wood-framed homes, the reader is referred to the NDS for symbol definitions, as well as other guidance.

5.3.1 Structural Safety Checks

Bending (Flexural) Capacity

The following equations from the NDS determine if a member has sufficient bending strength. Notches in bending members should be avoided, but small notches are permissible; refer to NDS•3.2.3. Similarly, the diameter of holes in bending members should not exceed one-third the member's depth and should be located along the center line of the member. Greater flexural capacity may be obtained by increasing member depth, decreasing the clear span or spacing of the member, or selecting a grade and species of lumber with a higher allowable bending stress. Engineered wood products or alternative materials may also be considered.

[NDS•3.3]

$f_b \le F_b'$ basic design check for bending stress

$F_b' = F_b \ x$ (applicable adjustment factors per Section 5.2.4)

$f_b = \frac{Mc}{I} = \frac{M}{S}$ extreme fiber bending stress due to bending moment from transverse load

$S = \frac{I}{c} = \frac{bd^2}{6}$ section modulus of rectangular member

$I = \frac{bd^3}{12}$ moment of inertia of rectangular member

$c = \frac{1}{2}d$ distance from extreme fiber to neutral axis

Horizontal Shear

Because shear parallel to grain (i.e., horizontal shear) is induced by bending action, it is also known as bending shear and is greatest at the neutral axis. Bending shear is not transverse shear; lumber will always fail in other modes before failing in transverse or cross-grain shear owing to the longitudinal orientation of the wood fibers in structural members.

The horizontal shear force is calculated for solid sawn lumber by including the component of all loads (uniform and concentrated) that act perpendicular to the bearing surface of the solid member in accordance with NDS•3.4.3. Loads within a distance, *d*, from the bearing point are not included in the horizontal shear calculation; *d* is the depth of the member for solid rectangular members. Transverse shear is not a required design check, although it is used to determine the magnitude of horizontal shear by using basic concepts of engineering mechanics as discussed below.

The following equations from NDS•3.4 for horizontal shear analysis are limited to solid flexural members such as solid sawn lumber, glulam, or mechanically laminated beams. Notches in beams can reduce shear capacity and should be considered in accordance with NDS•3.4.4. Also, bolted connections influence the shear capacity of a beam; refer to NDS•3.4.5. If required, greater horizontal shear capacity may be obtained by increasing member depth or width, decreasing the clear span or spacing of the member, or selecting another species with a higher allowable shear capacity. The general equation for horizontal shear stress is discussed in the NDS and in mechanics of materials text books. Because dimension lumber is solid and rectangular, the simple equation for f_v is most commonly used.

[NDS•3.4]

$f_v \le F'_v$	basic design check for horizontal shear
$F'_v = F_v$ x	(applicable adjustment factors per Section 5.2.4)
$f_v = \frac{VQ}{Ib}$	horizontal shear stress (general equation)
$f_v = \frac{3V}{2A}$	for maximum horizontal shear stress at the neutral axis of solid rectangular members

Compression Perpendicular to Grain (Bearing)

For bending members bearing on wood or metal, a minimum bearing of 1.5 inches is typically recommended. For bending members bearing on masonry, a minimum bearing of 3 inches is typically advised. The resulting bearing areas may not, however, be adequate in the case of heavily loaded members. On the other hand, they may be too conservative in the case of lightly loaded members. The minimum bearing lengths are considered to represent good practice.

The following equations from the NDS are based on net bearing area. Note that the provisions of the NDS acknowledge that the inner bearing edge experiences added pressure as the member bends. As a practical matter, the added pressure does not pose a problem because the compressive capacity, $F'_{c\perp}$, of wood increases as the material is compressed. Further, the design value is based

on a deformation limit, not on failure by crushing. Thus, the NDS recommends the added pressure at bearing edges not be considered. The designer is also alerted to the use of the bearing area factor, C_b, which accounts for the ability of wood to distribute large stresses originating from a small bearing area not located near the end of a member. Examples include interior bearing supports and compressive loads on washers in bolted connections.

[NDS•3.10]

$f_{c\perp} \le F'_{c\perp}$ basic design check for compression perpendicular to grain

$F'_{c\perp} = F_{c\perp}$ x (applicable adjustment factors per Section 5.2.4)

$f_{c\perp} = \frac{P}{A_b}$ stress perpendicular to grain due to load, P, on net bearing area, A_b.

The above equations pertain to bearing that is perpendicular to grain; for bearing at an angle to grain, refer to NDS•3.10. The later condition would apply to sloped bending members (i.e., rafters) notched at an angle for bearing. For light-frame construction, bearing stress is rarely a limiting factor.

Combined Bending and Axial Loading

Depending on the application and the combination of loads considered, some members such as wall studs and roof truss members, experience bending stress in addition to axial loading. The designer should evaluate combined bending and axial stresses as appropriate. If additional capacity is required, the selection of a higher grade of lumber is not always an efficient solution for overstressed compression members under combined axial and bending loads because the design may be limited by stability rather than by a stress failure mode. Efficiency issues will become evident when the designer calculates the components of the combined stress interaction equations that are given below and found in the NDS.

[NDS•3.9]

Combined bending and axial tension design check

$$\frac{f_t}{F'_t} + \frac{f_b}{F_b^*} \le 1$$

$$\frac{f_b - f_t}{F_b^{**}} \le 1$$

Combined bending and axial compression design check

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b1}}{F'_{b1}\left(1 - f_c / F_{cE1}\right)} + \frac{f_{b2}}{F'_{b2}\left(1 - \left(f_c / F_{cE2}\right) - \left(f_{b1} / F_{bE}\right)^2\right)} \le 1$$

Compression and Column Stability

For framing members that support axial loads only (i.e., columns), the designer must consider whether the framing member can withstand the axial compressive forces on it without buckling or compressive failure. If additional

compression strength is required, the designer should increase member size, decrease framing member spacing, provide additional lateral support, or select a different grade and species of lumber with higher allowable stresses. Improving lateral support is usually the most efficient solution when stability controls the design (disregarding any architectural limitations). The need for improved lateral support will become evident when the designer performs the calculations necessary to determine the stability factor, C_P, in accordance with NDS•3.7. When a column has continuous lateral support in two directions, buckling is not an issue and C_p = 1.0. If, however, the column is free to buckle in one or more directions, C_p must be evaluated for each direction of possible buckling. The evaluation must also consider the spacing of intermediate bracing, if any, in each direction.

[NDS•3.7]

$f_c \le F_c'$ basic design check for compression parallel to grain

$F_c' = F_c$ x (applicable adjustment factors from Section 5.2.4, including C_p)

$f_c = \frac{P}{A}$ compressive stress parallel to grain due to axial load, P, acting on the member's cross-sectional area, A.

$$C_p = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$ column stability factor

$$F_{cE} = \frac{K_{cE}E'}{\left(\ell_e/d\right)^2}$$

$F_c^* = F_c$ x (same adjustment factors for F_c' except C_p is not used)

Tension

Relatively few members in light-frame construction resist tension forces only. One notable exception occurs in roof framing where cross-ties or bottom chords in trusses primarily resist tension forces. Other examples include chord and collector members in shear walls and horizontal diaphragms as discussed in Chapter 6. Another possibility is a member subject to excessive uplift loads such as those produced by extreme wind. In any event, connection design is usually the limiting factor in designing the transfer of tension forces in light-frame construction (refer to Chapter 7). Tension stresses in wood members are checked by using the equations below in accordance with NDS•3.8.

[NDS•3.8]

$f_t \le F_t'$ basic design check for tension parallel to grain

$F_t' = F_t$ x (applicable adjustment factors per Section 5.2.4)

$f_t = \frac{P}{A}$ stress in tension parallel to gain due to axial tension load, P, acting on the member's cross-sectional area, A

The NDS does not provide explicit methods for evaluating cross-grain tension forces and generally recommends the avoidance of cross-grain tension in lumber even though the material is capable of resisting limited cross-grain stresses. Design values for cross-grain tension may be approximated by using one-third of the unadjusted horizontal shear stress value, F_v. One application of cross-grain tension in design is in the transfer of moderate uplift loads from wind through the band or rim joist of a floor to the construction below. If additional cross-grain tension strength is required, the designer should increase member size or consider alternative construction details that reduce cross-grain tension forces. When excessive tension stress perpendicular to grain cannot be avoided, the use of mechanical reinforcement or design detailing to reduce the cross-grain tension forces is considered good practice (particularly in high-hazard seismic regions) to ensure that brittle failures do not occur.

5.3.2 Structural Serviceability

Deflection Due to Bending

The NDS does not specifically limit deflection but rather defers to designer judgment or building code specifications. Nonetheless, with many interior and exterior finishes susceptible to damage by large deflections, reasonable deflection limits based on design loads are recommended herein for the design of specific elements.

The calculation of member deflection is based on the section properties of the beam from NDS-S and the member's modulus of elasticity with applicable adjustments. Generally, a deflection check using the equations below is based on the estimated maximum deflection under a specified loading condition. Given that wood exhibits time- and load-magnitude-dependent permanent deflection (creep), the total long-term deflection can be estimated in terms of two components of the load related to short- and long-term deflection using recommendations provided in NDS•3.5.

[NDS•3.5]

$$\Delta_{estimate} \leq \Delta_{allow} = \frac{\ell}{(120 \text{ to } 600)} \text{ (see Table 5.5 for value of denominator)}$$

$$\Delta_{estimate} \cong f\left(\frac{\text{load and span}}{EI}\right) \text{ (see beam equations in Appendix A)}$$

If a deflection check proves unacceptable, the designer may increase member depth, decrease the clear span or spacing of the member, or select a grade and species of wood with a higher modulus of elasticity (the least effective option). Typical denominator values used in the deflection equation range from 120 to 600 depending on application and designer judgment. Table 5.5 provides recommended deflection limits. Certainly, if a modest adjustment to a deflection limit results in a more efficient design, the designer should exercise discretion with respect to a possible negative consequence such as vibration or long-term creep. For lateral bending loads on walls, a serviceability load for a deflection check may be considered as a fraction of the nominal design wind load for

exterior walls. A reasonable serviceability wind load criteria may be taken as 0.75W or 75 percent of the nominal design wind load (Galambos and Ellingwood, 1986).

TABLE 5.5 Recommended Allowable Deflection Limits[1]

Element or Condition	Deflection Limit, Δ_{all} [2]	Load Condition
Rafters without attached ceiling finish	ℓ/180	L_r or S
Rafters with attached ceiling finishes and trusses	ℓ/240	L_r or S
Ceiling joists with attached finishes	ℓ/240	L_{attic}
Roof girders and beams	ℓ/240	L_r or S
Walls	ℓ/180	W or E
Headers	ℓ/240	(L_r or S) or L
Floors[3]	ℓ/360	L
Floor girders and beams[4]	ℓ/360	L

Notes:
[1]Values may be adjusted according to designer discretion with respect to potential increases or decreases in serviceability. In some cases, a modification may require local approval of a code variance. Some deflection checks may be different or not required depending on the local code requirements. The load condition includes the live or transient load only, not dead load.
[2]ℓ is the clear span in units of inches for deflection calculations.
[3]Floor vibration may be controlled by using ℓ/360 for spans up to 15 feet and a 1/2-inch limit for spans greater than 15 feet. Wood I-joist manufacturers typically recommend ℓ/480 as a deflection limit to provide enhanced floor performance and to control nuisance vibrations.
[4]Floor vibration may be controlled for combined girder and joist spans of greater than 20 feet by use of a ℓ/480 to ℓ/600 deflection limit for the girder.

Given that system effects influence the stiffness of assemblies in a manner similar to that of bending capacity (see Section 5.2.4.2), the system deflection factors of Table 5.6 are recommended. The estimated deflection based on an analysis of an element (e.g., stud or joist) is multiplied by the deflection factors to account for system effect. Typical deflection checks on floors under uniform loading can be easily overestimated by 20 percent or more. In areas where partitions add to the rigidity of the supporting floor, deflection can be overestimated by more than 50 percent (Hurst, 1965). When concentrated loads are considered on typical light-frame floors with wood structural panel subflooring, deflections can be overestimated by a factor of 2.5 to 3 due to the neglect of the load distribution to adjacent framing members and partial composite action (Tucker and Fridley, 1999). Similar results have been found for sheathed wall assemblies (NAHBRF, 1974). When adhesives attach wood structural panels to wood framing, even greater reductions in deflection are realized due to increased composite action (Gillespie et al., 1978; Pellicane and Anthony, 1996). However, if a simple deflection limit such as ℓ/360 is construed to control floor vibration in addition to the serviceability of finishes, the use of system deflection factors of Table 5.6 is not recommended for floor system design. In this case, a more accurate estimate of actual deflection may result in a floor with increased tendency to vibrate or bounce.

TABLE 5.6 System Deflection Adjustment Factors[1]

Framing System	Multiply single member deflection estimate by:
Light-wood-frame floor system with minimum 2x8 joists, minimum 3/4-inch-thick sheathing,[2] and standard fastening	0.85–Uniform load 0.4–Concentrated load
Light-wood-frame floor system as above, but with glued and nailed sheathing	0.75–Uniform load 0.35–Concentrated load
Light-wood-frame wall system with 2x4 or 2x6 studs with minimum 3/8-inch-thick sheathing on one side and 1 /2-inch-thick gypsum board on the other; both facings applied with standard fastening[3]	0.7–2x4 0.8–2x6

Notes:
[1]System deflection factors are not recommended when evaluating floor member deflection limits of Table 5.5 with the implied purpose of controlling floor vibration.
[2]Two sheathing layers may be used to make up a minimum thickness of 3/4-inch.
[3]The factors may be adjusted according to fastener diameter in accordance with footnote 5 of Table 5.4. If fastening is doubled (i.e., spacing halved), the factors may be divided by 1.4 (Polensek, 1975).

Floor Vibration

The NDS does not specifically address floor vibration because it is a serviceability rather than a safety issue. In addition, what is considered an "acceptable" amount of floor vibration is highly subjective. Accordingly, reliable design information on controlling floor vibration to meet a specific level of "acceptance" is not readily available; therefore, some rules of thumb are provided below for the designer wishing to limit vibration beyond that implied by the traditional use of an $\ell/360$ deflection limit (FHA, 1958; Woeste and Dolan, 1998).

- For floor joist spans less than 15 feet, a deflection limit of $\ell/360$ considering design live loads only may be used, where ℓ is the clear span of the joist in inches.
- For floor joist clear spans greater than 15 feet, the maximum deflection should be limited to 0.5 inches.
- For wood I-joists, the manufacturer's tables that limit deflection to $\ell/480$ should be used for spans greater than 15 feet, where ℓ is the clear span of the member in inches.
- When calculating deflection based on the above rules of thumb, the designer should use a 40 psf live load for all rooms whether or not they are considered sleeping rooms.
- As an additional recommendation, glue and mechanically fasten the floor sheathing to the floor joists to enhance the floor system's strength and stiffness.

Floor deflections are typically limited to $\ell/360$ in the span tables published in current building codes using a standard deflection check without consideration of system effects. For clear spans greater than 15 feet, this deflection limit has caused nuisance vibrations that are unacceptable to some building occupants or owners. Floor vibration is also aggravated when the floor is supported on a bending member (e.g., girder) rather than on a rigid bearing wall. It may be

desirable to design such girders with a smaller deflection limit to control floor vibration, particularly when girder and floor spans have more than a 20-foot total combined span (i.e., span of girder plus span of supported floor joist).

For metal-plate-connected wood trusses, strong-backs are effective in reducing floor vibration when they are installed through the trusses near the center of the span. A strong-back is a continuous bracing member, typically a 2x6, fastened edgewise to the base of the vertical web of each truss with 2-16d nails. For longer spans, strong-backs may be spaced at approximately 8-foot intervals across the span. Details for strong-backs may be found in the *Metal Plate Connected Wood Truss Handbook* (WTCA, 1997). Alternatively, a more stringent deflection criteria may be used for the floor truss design.

Shrinkage

The amount of wood shrinkage in a structure depends on the moisture content (MC) of the lumber at the time of installation relative to the equilibrium moisture content (EMC) that the wood will ultimately attain in use. It is also dependent on the detailing of the structure such as the amount of lumber supporting loads in a perpendicular-to-grain orientation (i.e., sill, sole, top plates, and joists). MC at installation is a function of the specified drying method, jobsite storage practices, and climate conditions during construction. Relatively dry lumber (15 percent or less) minimizes shrinkage problems affecting finish materials and prevents loosening or stressing of connections. A less favorable but acceptable alternative is to detail the structure such that shrinkage is uniform, dispersed, or otherwise designed to minimize problems. This alternative is the "defacto" choice in simple residential buildings.

Shrink and swell across the width or thickness of lumber can be estimated by the equation below from ASTM D1990 for typical softwood structural lumber (ASTM, 1998a). Shrinkage in the longitudinal direction of the member is practically negligible.

[ASTM D1990•App. X.1]

$$d_2 = d_1\left(\frac{1-\frac{a-0.2M_2}{100}}{1-\frac{a-0.2M_1}{100}}\right)$$

d_1 = member width or thickness at moisture content M_1
d_2 = member width or thickness at moisture content M_2

$a = 6.0$ (for width dimension)
$a = 5.1$ (for thickness dimension)

5.4 Floor Framing

The objectives of floor system design are

- to support occupancy live loads and building dead loads adequately;
- to resist lateral forces resulting from wind and seismic loads and to transmit the forces to supporting shear walls through diaphragm action;
- to provide a suitable subsurface for floor finishes;
- to avoid owner complaints (e.g., excessive vibration, noise, etc.);
- to serve as a thermal barrier over unconditioned areas (e.g., crawl spaces); and
- to provide a one- to two-hour fire rating between dwelling units in multifamily buildings (refer to local building codes).

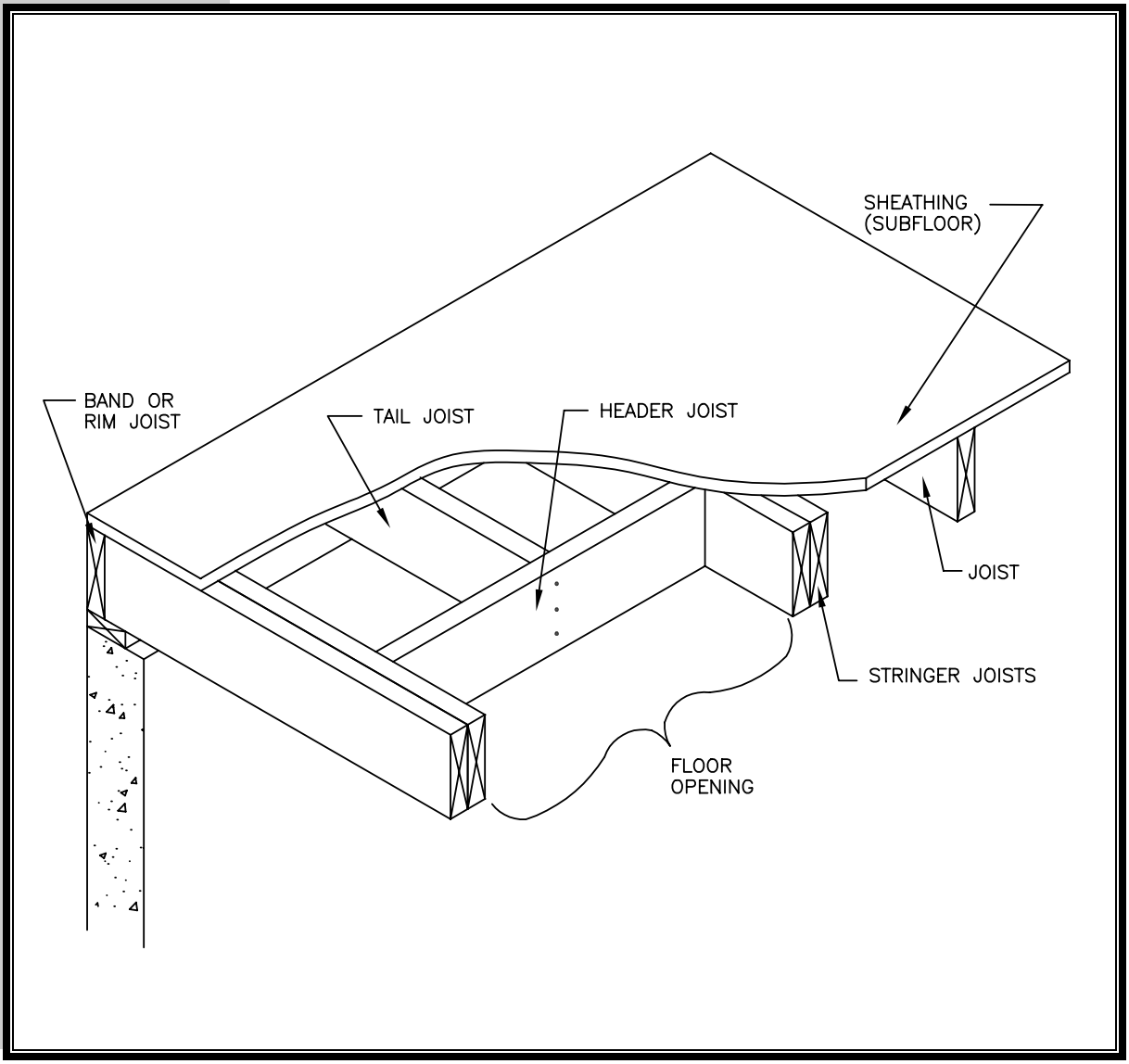
5.4.1 General

A wood floor is a horizontal structural system composed primarily of the following members:

- joists;
- girders; and
- sheathing.

Wood floor systems have traditionally been built of solid sawn lumber for floor joists and girders, although parallel chord wood trusses and wood I-joists are seeing increasing use, and offer advantages for dimensional consistency, and spans. Floor joists are horizontal, repetitive framing members that support the floor sheathing and transfer the live and dead floor loads to the walls, girders, or columns below. Girders are horizontal members that support floor joists not otherwise supported by interior or exterior load-bearing walls. Floor sheathing is a horizontal structural element, usually plywood or oriented strand board panels, that directly supports floor loads and distributes the loads to the framing system below. Floor sheathing also provides lateral support to the floor joists. As a structural system, the floor provides resistance to lateral building loads resulting from wind and seismic forces and thus constitutes a “horizontal diaphragm” (refer to Chapter 6). Refer to Figure 5.2 for an illustration of floor system structural elements and to *Cost-Effective Home Building: A Design and Construction Handbook* for efficient design ideas and concepts (NAHBRC, 1994).

FIGURE 5.2 Structural Elements of the Floor System



The design approach discussed herein addresses solid sawn lumber floor systems in accordance with the procedures specified in the *National Design Specification for Wood Construction* (NDS), with appropriate modifications as noted. For more information regarding wood I-joists, trusses, and other materials, consult the manufacturer's specifications and applicable code evaluation reports.

Section 5.3 discusses the general design equations and design checks for the NDS. The present section provides detailed design examples that apply the equations in Section 5.3, while tailoring them to the design of the elements in a floor system. The next sections make reference to the span of a member. The NDS defines span as the clear span of the member plus one-half the required bearing at each end of the member. This guide simply defines span as the clear span between bearing points.

When designing any structural element, the designer must first determine the loads acting on the element. Load combinations used in the analysis of floor

members in this guide are taken from Table 3.1 of Chapter 3. Given that only the dead loads of the floor system and live loads of occupancy are present in a typical floor system, the controlling design load combination for a simply-supported floor joist is D+L. For joists with more complicated loading, such as cantilevered joists supporting roof framing, the following load combinations may be considered in accordance with Chapter 3:

$D + L$
$D + L + 0.3 (L_r \text{ or } S)$
$D + (L_r \text{ or } S) + 0.3L$

5.4.2 Floor Joist Design

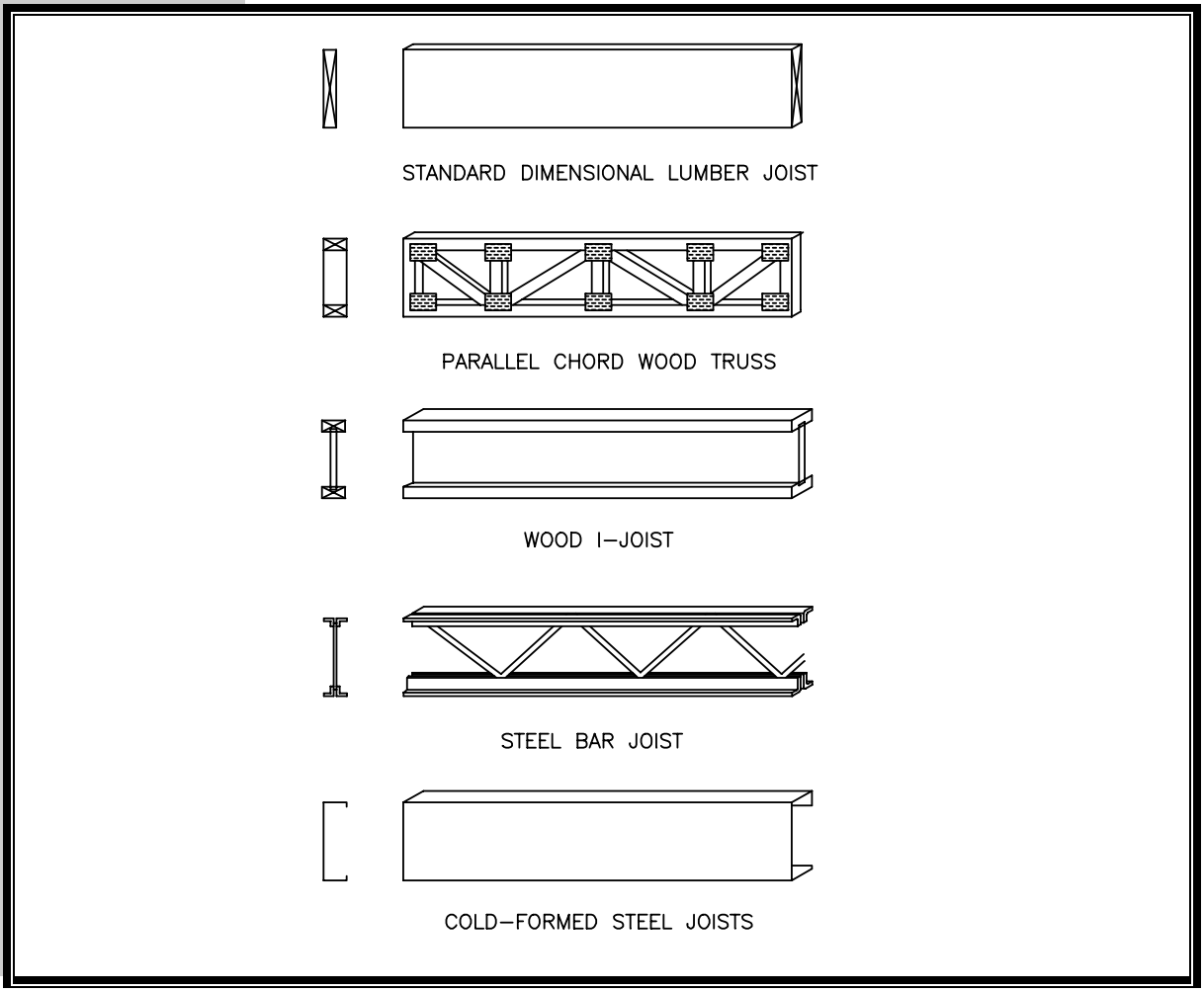
Readily available tables in residential building codes provide maximum allowable spans for different species, grades, sizes, and spacings of lumber joists. Some efficient concepts for floor joist design are also provided in *Cost Effective Home Building: A Design and Construction Handbook* (NAHB, 1994). Therefore, it is usually not necessary to design conventional floor joists for residential construction. To obtain greater economy or performance, however, designers may wish to create their own span tables or spreadsheets for future use in accordance with the methods shown in this section.

Keep in mind that the grade and species of lumber is often a regional choice governed by economics and availability; some of the most common species of lumber for floor joists are Hem-Fir, Spruce-Pine-Fir, Douglas-Fir, and Southern Yellow Pine. Bear in mind, too, that the most common sizes for floor joists are 2x8 and 2x10, although 2x12s are also frequently used. The following examples are located in Section 5.7 and illustrate the design of typical floor joists in accordance with the principles discussed earlier:

- simple span joist (Examples 5.1 and 5.2); and
- cantilevered joist (Example 5.3).

For different joist applications, such as a continuous multiple span, the designer should use the appropriate beam equations (refer to Appendix A) to estimate the stresses induced by the loads and reactions. Other materials such as wood I-joists and parallel chord floor trusses are also commonly used in light-frame residential and commercial construction; refer to the manufacturer's data for span tables for wood I-joists and other engineered wood products. For additional information on wood floor trusses that can be ordered to specification with engineering certification (i.e., stamped shop drawings), refer to Section 5.6.3 on roof trusses. Cold-formed steel floor joists or trusses may also be considered. Figure 5.3 illustrates some conventional and alternative floor joist members.

FIGURE 5.3 Conventional and Alternative Floor Framing Members



Notes:
[1]Trusses are also available with trimmable ends.
[2]Cold-formed steel is also used to make floor trusses.

For typical floor systems supporting a concentrated load at or near center span, load distribution to adjacent joists can substantially reduce the bending stresses or moment experienced by the loaded joist. A currently available design methodology may be beneficial for certain applications such as wood-framed garage floors that support heavy concentrated wheel loads (Tucker and Fridley, 1999). Under such conditions, the maximum bending moment experienced by any single joist is reduced by more than 60 percent. A similar reduction in the shear loading (and end reaction) of the loaded joist also results, with exception for "moving" concentrated loads that may be located near the end of the joist, thus creating a large transverse shear load with a small bending moment. The above-mentioned design methodology for a single, concentrated load applied near mid-span of a repetitive member floor system is essentially equivalent to using a C_r factor of 1.5 or more (see Section 5.2.4.2). The system deflection adjustment factors in Table 5.6 are applicable as indicated for concentrated loads.

Bridging or cross-braces were formerly thought to provide both necessary lateral-torsional bracing of dimension lumber floor joists and stiffer floor systems.

However, full-scale testing of 10 different floor systems as well as additional testing in completed homes has conclusively demonstrated that bridging or cross-bracing provides negligible benefit to either the load-carrying capacity or stiffness of typical residential floors with dimension lumber framing (sizes of 2x6 through 2x12) and wood structural panel subflooring (NAHB, 1961). These same findings are not proven to apply to other types of floor joists (i.e., I-joists, steel joists, etc.) or for dimension lumber joists greater than 12 inches in depth. According to the study, bridging may be considered necessary for 2x10 and 2x12 dimension lumber joists with clear spans exceeding about 16 feet and 18 feet, respectively (based on a 50 psf total design load and L/360 deflection limit). To the contrary, the beam stability provisions of NDS•4.4.1 conservatively require bridging to be spaced at intervals not exceeding 8 feet along the span of 2x10 and 2x12 joists.

5.4.3 Girder Design

The decision to use one girder over another is a function of cost, availability, span and loading conditions, clearance or head-room requirements, and ease of construction. Refer to the Figure 5.4 for illustrations of girder types. Girders in residential construction are usually one of the following types:

- built-up dimension lumber;
- steel I-beam;
- engineered wood beam;
- site-fabricated beam;
- wood I-joist; or
- metal plate connected wood truss.

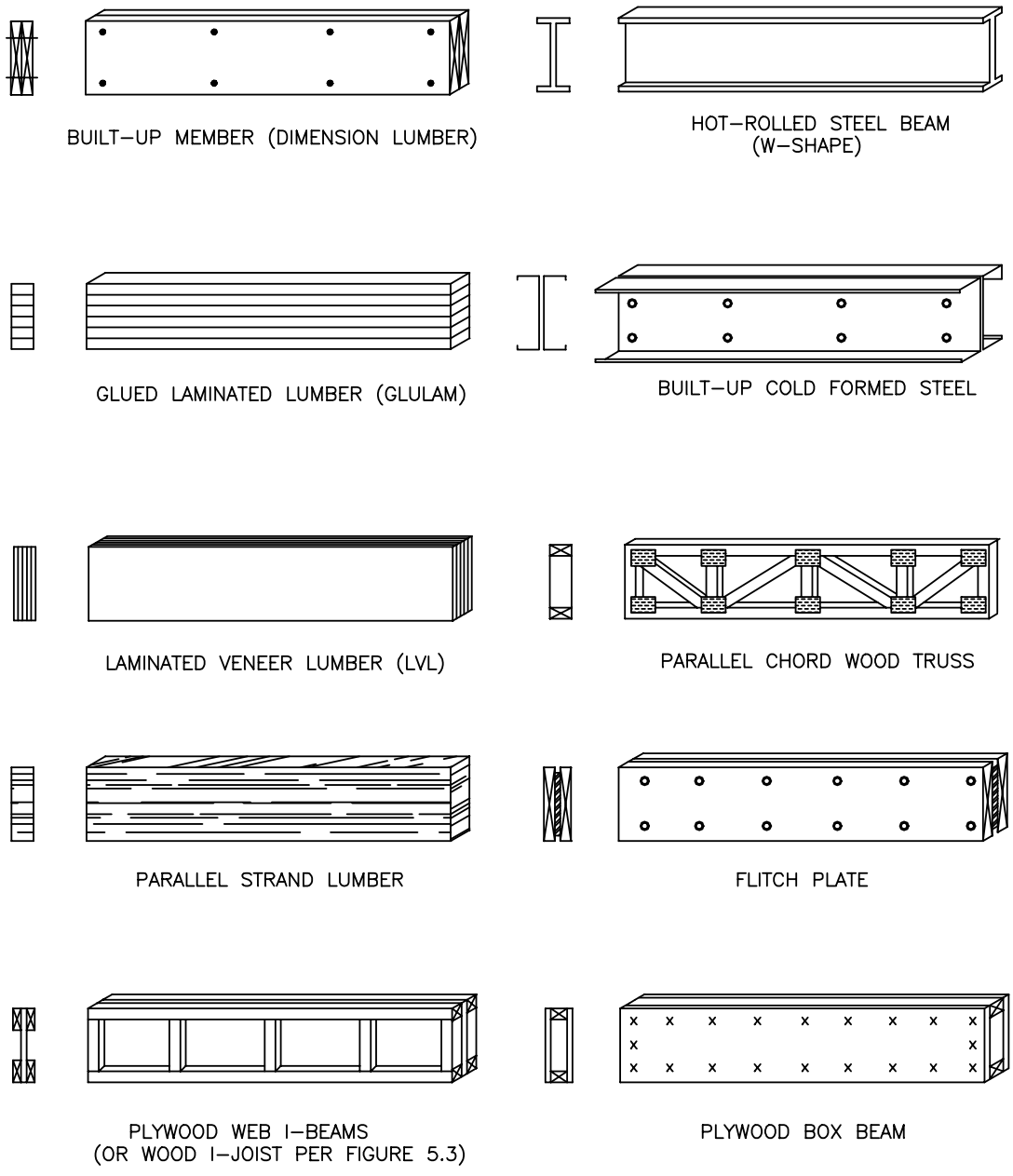
Built-up beams are constructed by nailing together of two or more plys of dimension lumber. Since load sharing occurs between the plys (i.e., lumber members), the built-up girder is able to resist higher loads than a single member of the same overall dimensions. The built-up member can resist higher loads only if butt joints are located at or near supports and are staggered in alternate plys. Each ply may be face nailed to the previous ply with 10d nails staggered at 12 inches on center top to bottom. The design method and equations are the same as those in Section 5.4.2 for floor joists; however, the adjustment factors applying to design values and loading conditions are somewhat different. The designer needs to keep the following in mind:

- Although floor girders are not typically thought of as “repetitive” members, a repetitive member factor is applicable if the floor girder is built-up from two or more members (three or more according to the NDS).

- The beam stability factor, C_L, is determined in accordance with NDS•3.3.3; however, for girders supporting floor framing, lateral support is considered to be continuous and $C_L = 1$.

Example 5.4 illustrates the design of a built-up floor girder.

FIGURE 5.4 Examples of Beams and Girders



Steel I beams are often used in residential construction because of their greater spanning capability. Compared with wood members, they span longer distances with a shallower depth. A 2x4 or 2x6 is usually attached to the top surface with bolts to provide a fastening surface for floor joists and other structural members. Although steel beam shapes are commonly referred to as I-beams, a typical 8-inch-deep W-shaped beam is commonly considered a house beam. Alternatively, built-up cold-formed steel beams (i.e., back-to-back C-shapes) may be used to construct I-shaped girders. Refer to the *Steel Construction Manual* (AISC, 1989) and the American Iron and Steel Institute's publication RG-936 for the design of and span tables for residential applications of hot-rolled steel sections (AISI, 1993). Structural steel floor beam span tables are also found in the *Beam Series* (NAHBRC, 1981). The *Prescriptive Method for Cold-Formed Steel in Residential Construction* should be consulted for the design of built-up cold-formed steel sections as headers and girders (NAHBRC, 1998).

Engineered wood beams include I-joists, wood trusses (i.e., girder trusses) glue-laminated lumber, laminated veneer lumber, parallel strand lumber, etc. This guide does not address the design of engineered wood girders because product manufacturers typically provide span tables or engineered designs that are considered proprietary. Consult the manufacturer for design guidelines or completed span tables. The NDS does, however, provide a methodology for the design of glue-laminated beams (NDS•5).

Site-fabricated beams include plywood box beams, plywood I-beams, and flitch plate beams. *Plywood box beams* are fabricated from continuous dimension lumber flanges (typically 2x4s or 2x6s) sandwiched between two plywood webs; stiffeners are placed at concentrated loads, end bearing points, plywood joints, and maximum 24-inch intervals. *Plywood I-beams* are similar to box beams except that the plywood web is sandwiched between dimension lumber wood flanges (typically 2x4s or 2x6s), and stiffeners are placed at maximum 24-inch intervals. *Flitch plate beams* are fabricated from a steel plate sandwiched between two pieces of dimension lumber to form a composite section. Thus, a thinner member is possible in comparison to a built-up wood girder of similar strength. The steel plate is typically 1/4 to 1/2 inches thick and about 1/4-inch less in depth than the dimension lumber. The sandwich construction is usually assembled with through-bolts staggered at about 12 inches on center. Flitch plate beams derive their strength and stiffness from the composite section of steel plate and dimension lumber. The lumber also provides a medium for fastening other materials using nails or screws.

Span tables for plywood I-beams, plywood box beams, steel-wood I-beams, and flitch plate beams are provided in NAHB's *Beam Series* publications (NAHBRC, 1981). Refer to the APA's *Product Design Specification* (PDS) and *Supplement* for the design method used for plywood box beams (APA, 1998b). The *International One- and Two-Family Dwelling Code* (ICC, 1998), formerly the *CABO One- and Two-Family Dwelling Code*, provides a simple prescriptive table for plywood box beam headers.

5.4.4 Subfloor Design

Typical subfloor sheathing is nominal 5/8- or 3/4-inch-thick 4x8 panels of plywood or oriented strand board (OSB) with tongue-and-groove edges at unsupported joints perpendicular to the floor framing. Sheathing products are generally categorized as wood structural panels and are specified in accordance with the prescriptive span rating tables published in a building code or are made available by the manufacturer. Example 5.5 uses the *Design and Construction Guide: Residential and Commercial* (APA, 1998a) to specify sheathing. The prescriptive tables provide maximum spans (joist spacing) based on sheathing thickness and span rating. It is important to note that the basis for the prescriptive tables is the standard beam calculation. If loads exceed the limits of the prescriptive tables, the designer may be required to perform calculations; however, such calculations are rarely necessary. In addition, the APA offers a plywood floor guide for residential garages that assists in specifying plywood subflooring suitable for heavy concentrated loads from vehicle tire loading (APA, 1980).

The APA also recommends a fastener schedule for connecting sheathing to floor joists. Generally, nails are placed a minimum of 6 inches on center at edges and 12 inches on center along intermediate supports. Refer to Table 5.7 for recommended nail sizes based on sheathing thickness. Nail sizes vary with nail type (e.g., sinkers, box nails, and common nails), and various nail types have different characteristics that affect structural properties (refer to Chapter 7). For information on other types of fasteners, consult the manufacturer. In some cases, shear loads in the floor diaphragm resulting from lateral loads (i.e., wind and earthquake) may require a more stringent fastening schedule; refer to Chapter 6 for a discussion on fastening schedules for lateral load design. Regardless of fastener type, gluing the floor sheathing to the joists increases floor stiffness and strength.

TABLE 5.7 Fastening Floor Sheathing to Structural Members[1]

Thickness	Size and Type of Fastener
Plywood and wood structural panels, subfloor sheathing to framing	
1/2-inch and less	6d nail
19/32- to 1-inch	8d nail
1-1/8- to 1-1/4-inch	10d nail or 8d deformed shank nail
Plywood and wood structural panels, combination subfloor/underlayment to framing	
3/4-inch and less	8d nail or 6d deformed shank nail
7/8- to -inch	8d nail
1-1/8- to 1-1/4-inch	10d nail or 8d deformed shank nail

Notes:
[1]Codes generally require common or box nails; if pneumatic nails are used, as is common, refer to NER-272 (NES, 1997) or the nail manufacturer's data. Screws are also commonly substituted for nails. For more detail on fasteners and connections, refer to Chapter 7.

While not as common today, boards may also be used as a subfloor (i.e., board sheathing). Floor sheathing boards are typically 1x6 or 1x8 material laid flatwise and diagonally (or perpendicular) on the floor joists. They may be designed using the NDS or local accepted practice.

5.5 Wall Framing

The objectives of wall system design are

- to resist snow, live, and dead loads and wind and seismic forces;
- to provide an adequate subsurface for wall finishes and to provide openings for doors and windows;
- to serve as a thermal and weather barrier;
- to provide space and access for electrical and mechanical equipment, where required; and
- to provide a one- to two-hour fire barrier if the wall separates individual dwelling units in attached or multifamily buildings.

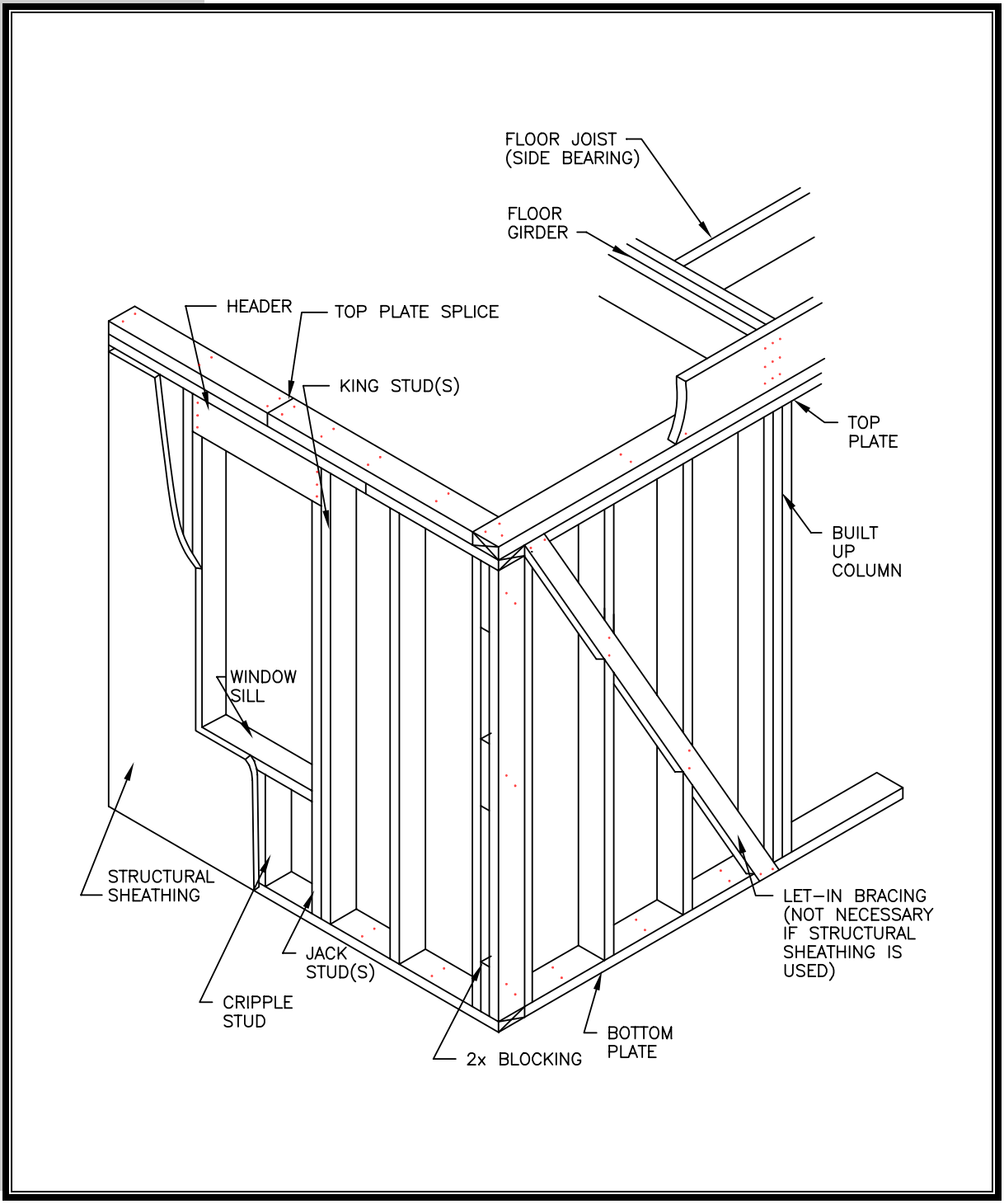
5.5.1 General

A wall is a vertical structural system that supports gravity loads from the roof and floors above and transfers the loads to the foundation below. It also resists lateral loads resulting from wind and earthquakes. A typical wood-framed wall is composed of the following elements as shown in Figure 5.5:

- studs, including wall, cripple, jack, and king studs;
- top and bottom (sole) plates;
- headers;
- sheathing; and
- diagonal let-in braces, if used.

Residential wall systems have traditionally been constructed of dimension lumber, usually 2x4s or 2x6s, although engineered wood studs and cold-formed steel studs are now seeing increased use. Wall studs are vertical, repetitive framing members spaced at regular intervals to support the wall sheathing. They span the full height of each story and support the building loads above. King and jack studs (also known as jamb studs) frame openings and support loads from a header. Cripple studs are placed above or below a wall opening and are not full height. Built-up wall studs that are assembled on the jobsite may be used within the wall to support concentrated loads. Top and bottom plates are horizontal members to which studs are fastened. The top and bottom plates are then fastened to the floor or roof above and either to the floor below or directly to the foundation. Headers are beams that transfer the loads above an opening to jack studs at each side of the opening.

FIGURE 5.5 Structural Elements of the Wall System



Structural wall sheathing, such as plywood or oriented strand board, distributes lateral loads to the wall framing and provides lateral support to both the wall studs (i.e., buckling resistance) and the entire building (i.e., racking resistance). Interior wall finishes also provide significant support to the wall studs and the structure. In low-wind and low-hazard seismic areas, metal 'T' braces or wood let-in braces may be used in place of wall sheathing to provide resistance to

lateral (i.e., racking) loads. About 50 percent of new homes constructed each year now use wood structural panel braces, and many of those homes are fully-sheathed with wood structural panels. These bracing methods are substantially stronger than the let-in brace approach; refer to Chapter 6 for greater detail on the design of wall bracing. Wood let-in braces are typically 1x4 wood members that are "let-in" or notched into the studs and nailed diagonally across wall sections at corners and specified intervals. Their use is generally through application of conventional construction provisions found in most building codes for residential construction in combination with interior and exterior claddings.

The design procedure discussed herein addresses dimension lumber wall systems according to the *National Design Specification for Wood Construction* (NDS). Where appropriate, modifications to the NDS have been incorporated and are noted. Standard design equations and design checks for the NDS procedure were presented earlier in this chapter. The detailed design examples in this section illustrate the application of the equations by tailoring them to the design of the elements that make up residential wall systems.

Wall systems are designed to withstand dead and live gravity loads acting parallel to the wall stud length, as well as lateral loads–primarily wind and earthquake loads–acting perpendicular to the face of the wall. Wind also induces uplift loads on the roof; when the wind load is sufficient to offset dead loads, walls and internal connections must be designed to resist tension or uplift forces. The outcome of the design of wall elements depends on the degree to which the designer uses the "system strength" inherent in the construction. To the extent possible, guidance on system design in this section uses the NDS and the recommendations in Sections 5.2 and 5.3.

When designing wall elements, the designer needs to consider the load combinations discussed in Chapter 3, particularly the following ASD combinations of dead, live, snow, and wind loads:

- $D + L + 0.3 (L_r \text{ or } S)$
- $D + (L_r \text{ or } S) + 0.3 L$
- $D + W$
- $D + 0.7E + 0.5L + 0.2S$

A wall system may support a roof only or a roof and one or more stories above. The roof may or may not include an attic storage live load. A 10 psf attic live load used for the design of ceiling joists is intended primarily to provide safe access to the attic, not storage. The controlling load combination for a wall that supports only a roof is the second load combination listed above. If the attic is not intended for storage, the value for *L* should be 0. The controlling load combination for a wall that supports a floor, wall, and a roof should be either the first or second load combination depending on the relative magnitude of floor and roof snow loads.

The third load combination provides a check for the out-of-plane bending condition due to lateral wind loads on the wall. For tall wood-frame walls that support heavy claddings such as brick veneer, the designer should also consider out-of-plane bending loads resulting from an earthquake load combination, although the other load combinations above usually control the design. The third

and fourth load combinations are essentially combined bending and axial loads that may govern stud design as opposed to axial load only in the first two load combinations. Chapter 6 addresses the design of walls for in-plane shear or racking forces resulting from lateral building loads caused by wind or earthquakes.

In many cases, certain design load combinations or load components can be dismissed or eliminated through practical consideration and inspection. They are a matter of designer judgment, experience, and knowledge of the critical design conditions.

5.5.2 Load-Bearing Walls

Exterior load-bearing walls support both axial and lateral loads. For interior load-bearing walls, only gravity loads are considered. A serviceability check using a lateral load of 5 psf is sometimes applied independently to interior walls but should not normally control the design of load-bearing framing. This section focuses on the axial and lateral load-bearing capacity of exterior and interior walls.

Exterior walls are not necessarily load-bearing walls. Load-bearing walls support gravity loads from either the roof, ceiling, or floor joists or the beams above. A gable-end wall is typically considered to be a nonload-bearing wall in that roof and floor framing generally runs parallel to the gable end; however, it must support lateral wind and seismic loads and even small dead and live loads. Exterior load-bearing walls must be designed for axial loads as well as for lateral loads from wind or seismic forces. They must also act as shear walls to resist racking loads from lateral wind or seismic forces on the overall building (refer to Chapter 6). Example 5.6 demonstrates the design of an exterior bearing wall.

When calculating the column stability factor for a stud wall, note that column capacity is determined by using the slenderness ratio about the strong axis of the stud $(l_e/d)_x$ in accordance with NDS•3.7.1. The reason for using the strong axis slenderness ratio is that lateral support is provided to the stud by the wall sheathing and finish materials in the stud's weak-axis bending or buckling direction. When determining the column stability factor, C_p, for a wall system rather than for a single column in accordance with NDS•3.7.1, the designer must exercise judgment with respect to the calculation of the effective length, ℓ_e, and the depth or thickness of the wall system, d. A buckling coefficient, K_e, of about 0.8 is reasonable (see Appendix G of NDS) and is supported in the research literature on this topic for sheathed wall assemblies and studs with square-cut ends (i.e., not a pinned joint).

In cases where continuous support is not present (e.g., during construction), the designer may want to consider stability for both axes. Unsupported studs generally fail due to weak-axis buckling under a significantly lower load than would otherwise be possible with continuous lateral support in the weak-axis buckling direction.

Interior walls may be either load-bearing or nonload-bearing. Nonload-bearing interior walls are often called partitions (see Section 5.5.3). In either case, interior walls should be solidly fastened to the floor and ceiling framing and to the exterior wall framing where they abutt. It may be necessary to install extra studs,

blocking, or nailers in the outside walls to provide for attachment of interior walls. The framing must also be arranged to provide a nailing surface for wallcovering materials at inside corners. For efficient construction details and concepts related to wall framing, refer to *Cost Effective Home Building: A Design and Construction Handbook* (NAHB, 1994).

Interior load-bearing walls typically support the floor or ceiling joists above when the clear span from exterior wall to exterior wall is greater than the spanning capability of the floor or ceiling joists. Interior walls, unlike exterior walls, seldom experience large transverse (i.e., out of plane) lateral loads; however, some building codes require interior walls to be designed for a minimum lateral load, such as 5 psf, for serviceability. If the interior wall is required only to resist axial loads, the designer may follow the design procedure demonstrated in Example 5.6 for the axial-load-only case. Generally, axial load design provides more-than-adequate resistance to a nominal lateral load.

If local code requirements do require wall studs to be designed to withstand a minimum lateral load, the designer should design load-bearing walls in accordance with the previous section on exterior load bearing walls. (Note that the load duration factor, C_D, of 1.6 is used for exterior load bearing walls when wind or earthquake loads are considered, whereas a load duration factor of 1.0 to 1.25 may be used for interior load-bearing walls and exterior walls analyzed for live and snow loads; refer to Section 5.2.4.1.)

5.5.3 NonLoad-Bearing Partitions

Interior partitions are not intended to support structural loads. Standard 2x4 or 2x3 wood stud interior partition walls are well proven in practice and do not require analysis. Openings within partitions do not require headers or trimmers and are commonly framed with single studs and horizontal members of the same size as the studs. Particularly in the case of closets, or other “tight” spaces, builders may frame certain partitions with smaller lumber, such as 2x2 studs or 2x4 studs turned flatwise to save space.

Where a minimum 5 psf lateral load check for serviceability is required in a nonload-bearing partition, the stud may be designed as a bending member or system similar to a simply supported floor joist, except that the only load is a 5 psf load uniformly distributed. The design approach and system factors in Sections 5.2 and 5.3 apply as appropriate.

5.5.4 Headers

Load-bearing headers are horizontal members that carry loads from a wall, ceiling, or floor or roof above and transfer the combined load to jack and king studs on each side of a window or door opening. The span of the header may be taken as the width of the rough opening measured between the jack studs supporting the ends of the header. Headers are usually built up from two nominal 2-inch-thick members.

Load-bearing header design and fabrication is similar to that for girders (see Section 5.4.3). This guide considers headers consisting of double members to be repetitive members; therefore, a repetitive member factor, C_r, of 1.1 to 1.2

should apply (refer to Table 5.4), along with a live load deflection limit of $\ell/240$ (refer to Table 5.6). Large openings or especially heavy loads may require stronger members such as engineered wood beams, hot-rolled steel, or flitch plate beams. Refer to *Cost-Effective Home Building: A Design and Construction Handbook* for economical framing solutions to reduce header loads and sizes (NAHB, 1994).

Headers are generally designed to support all loads from above; however, typical residential construction calls for a double top plate above the header. When an upper story is supported, a floor band joist and sole plate of the wall above are also spanning the wall opening below. These elements are all part of the resisting system. Recent header testing determined whether an adjustment factor (i.e., system factor or repetitive member factor) is justified in designing a header (HUD, 1999). The results showed that a repetitive member factor is valid for headers constructed of only two members as shown in Table 5.4 and that additional system effects produce large increases in capacity when the header is overlaid by a double top plate, band joist and sole plate as shown in Example 5.7. Consequently, an overall system factor of 1.8 was found to be a simple, conservative design solution. That system factor is applicable to the adjusted bending stress value, F_b', of the header member only. While this example covers only a very specific condition, it exemplifies the magnitude of potential system effect in similar conditions. In this case, the system effect is associated with load sharing and partial composite action. The above adjustment factor is not currently recognized in the NDS.

Refer to Table 5.8 for recommended allowable bending stress adjustment factors for use in the specific header design conditions related to the discussion above. For other conditions, refer to Table 5.4. Example 5.7 demonstrates the design approach for a typical header condition.

TABLE 5.8 Recommended System Adjustment Factors for Header Design

Header Type and Application[1]	Recommended C_r Value[2]
2x10 double header of No. 2 Spruce-Pine-Fir	1.30[3]
Above header with double top plate, 2x10 floor band joist, and sole plate of wall located directly above.[4]	1.8

Notes:
[1]For other applications and lumber sizes or grades, refer to the C_r factors in Table 5.4 of Section 5.2.4.2.
[2]Apply C_r in lieu of Section 5.1.3 (Table 5.4) to determine adjusted allowable bending stress, F_b'.
[3]Use $C_r = 1.35$ when the header is overlaid by a minimum 2x4 double top plate without splices.
[4]Refer to Example 5.7 for an illustration of the header system.

Headers are not required in nonload-bearing walls. Openings can be framed with single studs and a horizontal header block of the same size. It is common practice to use a double 2x4 or triple 2x4 header for larger openings in nonload-bearing walls. In the interest of added rigidity and fastening surface, however, some builders use additional jamb studs for openings in nonload-bearing walls, but such studs are not required.

5.5.5 Columns

Columns are vertical members placed where an axial force is applied parallel to the longitudinal axis. Columns may fail by either crushing or buckling. Longer columns have a higher tendency than shorter columns to fail due to buckling. The load at which the column buckles (Euler buckling load) is directly related to the ratio of the column's unsupported length to its depth (slenderness factor). The equations provided in Section 5.3 are based on the NDS•3.7.1 provisions regarding the compression and stability of an axial compression member (i.e., column) and thus account for the slenderness factor.

Figure 5.6 illustrates three ways to construct columns using lumber. *Simple columns* are columns fabricated from a single piece of sawn lumber; *spaced columns* are fabricated from two or more individual members with their longitudinal axes parallel and separated with blocking at their ends and mid-point(s); *built-up columns* are solid columns fabricated from several individual members fastened together. Spaced columns as described in the NDS are not normally used in residential buildings and are not addressed here (refer to NDS•15.2 for the design of spaced columns).

Steel jack posts are also commonly used in residential construction; however, jack post manufacturers typically provide a rated capacity so that no design is required except the specification of the design load requirements and the selection of a suitable jack post that meets or exceeds the required loading. Typical 8-foot tall steel jack posts are made of pipe and have adjustable bases for floor leveling. The rated (design) capacity generally ranges from 10,000 to 20,000 lbs depending on the steel pipe diameter and wall thickness.

Simple columns are fabricated from one piece of sawn lumber. In residential construction, simple columns such as a 4x4 are common. The equations in Section 5.3 are used to design simple columns as demonstrated in Example 5.8.

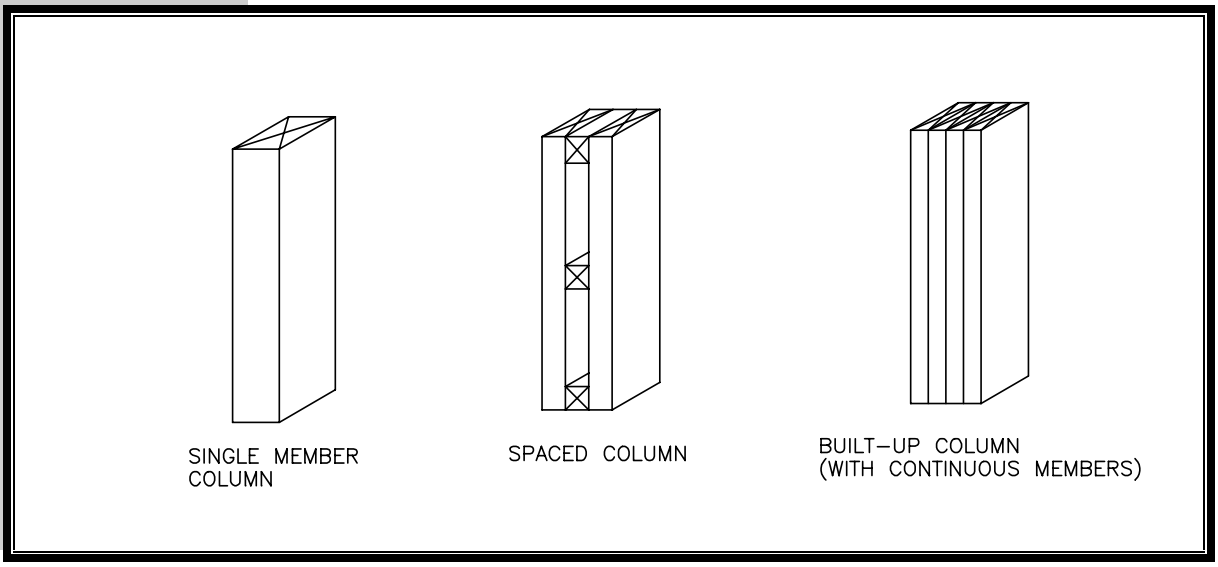
Built-up columns are fabricated from several wood members fastened together with nails or bolts. They are commonly used in residential construction because smaller members can be easily fastened together at the jobsite to form a larger column with adequate capacity.

The nails or bolts used to connect the plys (i.e., the separate members) of a built-up column do not rigidly transfer shear loads; therefore, the bending load capacity of a built-up column is less than a single column of the same species, grade, and cross-sectional area when bending direction is perpendicular to the laminations (i.e., all members bending in their individual weak-axis direction). The coefficient, K_f, accounts for the capacity reduction in bending load in nailed or bolted built-up columns. It applies, however, only to the weak-axis buckling or bending direction of the individual members and therefore should not be used to determine C_p for column buckling in the strong-axis direction of the individual members. (Refer to NDS•15.3 for nailing and bolting requirements for built-up columns.)

The above consideration is not an issue when the built-up column is sufficiently braced in the weak-axis direction (i.e., embedded in a sheathed wall assembly). In this typical condition, the built-up column is actually stronger than a solid sawn member of equivalent size and grade because of the repetitive member

effect on bending capacity (see Table 5.4). However, when the members in the built-up column are staggered or spliced, the column bending strength is reduced. While the NDS•15.3 provisions apply only to built-up columns with all members extending the full height of the column, design methods for spliced columns are available (ASAE, 1997).

FIGURE 5.6 Wood Column Types



5.6 Roofs

The objectives of roof framing design are

- to support building dead and snow loads and to resist wind and seismic forces;
- to resist roof construction and maintenance loads;
- to provide a thermal and weather barrier;
- to provide support for interior ceiling finishes; and
- to provide attic space and access for electrical and mechanical equipment or storage.

5.6.1 General

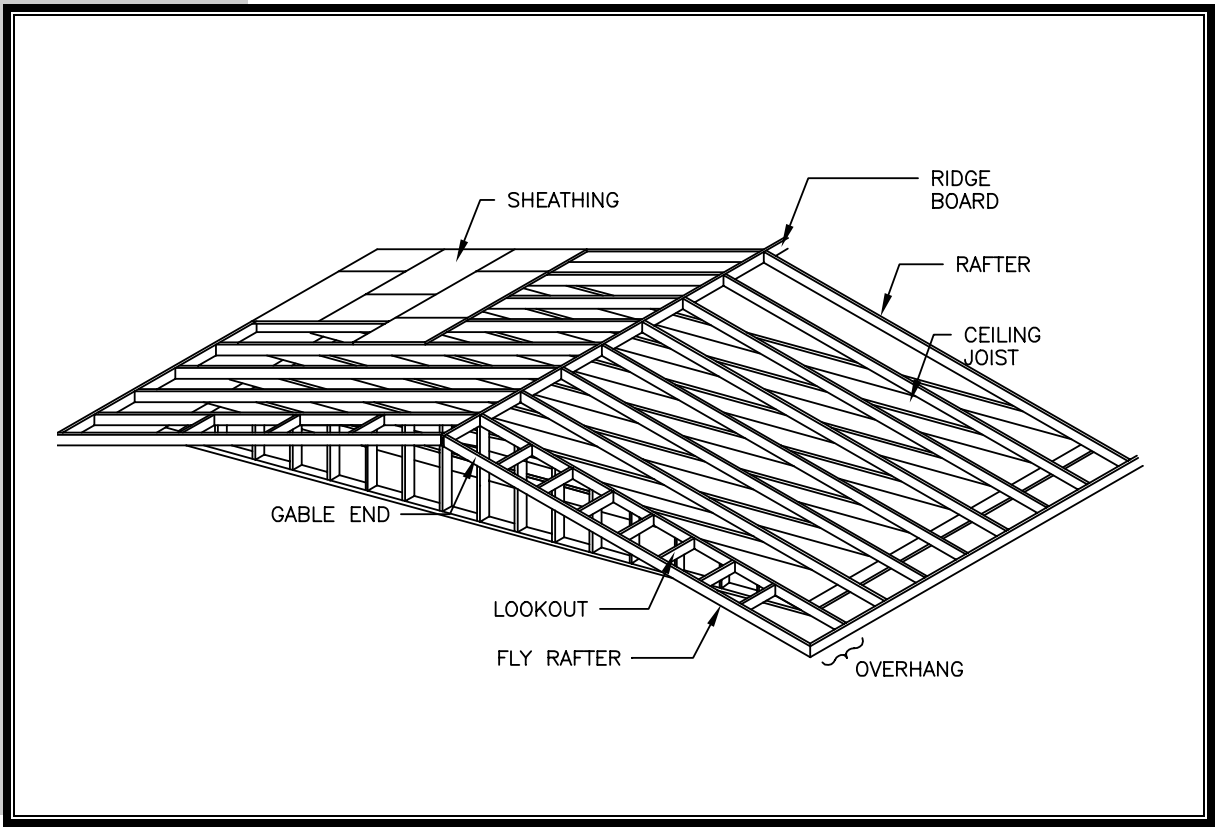
A roof in residential construction is typically a sloped structural system that supports gravity and lateral loads and transfers the loads to the walls below. Generally, the four options for wood roof construction are

- roof trusses;
- rafters and cross-ties;
- rafters with ridge beams (i.e. cathedral ceiling); and
- timber framing.

By far the most common types of residential roof construction use light-frame trusses, rafters, or a mix of these depending on roof layout. Figure 5.7 depicts conventional roof construction and roof framing elements. Rafters are repetitive framing members that support the roof sheathing and typically span from the exterior walls to a nonstructural ridge board (i.e., reaction plate). Rafter pairs may also be joined at the ridge with a gusset, thereby eliminating the need for a ridge board. Rafters may also be braced at or near mid-span using intermittent 2x vertical braces and a 2x runner crossing the bottom edges of the rafters. Ceiling joists are repetitive framing members that support ceiling and attic loads and transfer the loads to the walls and beams below. They are not normally designed to span between exterior walls and therefore require an intermediate bearing wall. Overhangs, where used, are framed extensions of the roof that extend beyond the exterior wall of the home, typically by 1 to 2 feet. Overhangs protect walls and windows from direct sun and rain and therefore offer durability and energy efficiency benefits.

Ceiling joists are typically connected to rafter pairs to resist outward thrust generated by loading on the roof. Where ceiling joists or cross-ties are eliminated to create a cathedral ceiling, a structural ridge beam must be used to support the roof at the ridge and to prevent outward thrust of the bearing walls. Ceiling joists and roof rafters are bending members that are designed similarly; therefore, this chapter groups them under one section.

FIGURE 5.7 Structural Elements of a Conventional Roof System



Roof trusses are preengineered components. They are fabricated from 2-inch-thick dimension lumber connected with metal truss plates. They are generally more efficient than stick framing and are usually designed to span from exterior wall to exterior wall with no intermediate support. In more complex portions of roof systems, it is still common to use rafter framing techniques.

Roof sheathing is a thin structural element, usually plywood or oriented strand board, that supports roof loads and distributes lateral and axial loads to the roof framing system. Roof sheathing also provides lateral support to the roof framing members and serves as a membrane or diaphragm to resist and distribute lateral building loads from wind or earthquakes (refer to Chapter 6).

Roof systems are designed to withstand dead, live, snow, and wind uplift loads; in addition, they are designed to withstand lateral loads, such as wind and earthquake loads, transverse to the roof system. The design procedure discussed herein addresses dimension lumber roof systems designed according to the NDS. Where appropriate, the procedure incorporates modifications of the NDS. Section 5.3 summarizes the general design equations and design checks based on the NDS. Refer to Chapter 6 for the design of roofs with respect to lateral loads on the overall structure; refer to Chapter 7 for guidance on the design of connections.

When designing roof elements or components, the designer needs to consider the following load combinations from Chapter 3 (Table 3.1):

- $D + (L_r \text{ or } S)$
- $0.6\,D + W_u$
- $D + W$

The following sections refer to the span of the member. The NDS defines span as the clear span of the member plus one-half the required bearing at each end of the member. For simplicity, the clear span between bearing points is used herein.

Finally, roofs exhibit system behavior that is in many respects similar to floor framing (see Section 5.4); however, sloped roofs also exhibit unique system behavior. For example, the sheathing membrane or diaphragm on a sloped roof acts as a folded plate that helps resist gravity loads. The effect of the folded plate becomes more pronounced as roof pitch becomes steeper. Such a system effect is usually not considered in design but explains why light wood-framed roof systems may resist loads several times greater than their design capacity. Recent research on trussed roof assemblies with wood structural panel sheathing points to a system capacity increase factor of 1.1 to 1.5 relative to the design of an individual truss (Wolfe and LaBissoniere, 1991; Wolfe, 1996; Mtenga, 1998). Thus, a conservative system factor of 1.15 is recommended in this document for chord bending stresses and a factor of 1.1 for chord tension and compression stresses.

5.6.2 Conventional Roof Framing

This section addresses the design of conventional roof rafters, ceiling joists (cross-ties), ridge beams, and hip and valley rafters. The design procedure for a rafter and ceiling joist system is similar to that of a truss, except that the

assembly of components and connections is site-built. It is common practice to use a standard pin-joint analysis to determine axial forces in the members and shear forces at their connections. The ceiling joists and rafters are then usually sized according to their individual applied bending loads taking into account that the axial load effects on the members themselves can be dismissed by judgment based on the large system effects in sheathed roof construction. Frequently, intermediate rafter braces that are similar to truss web members are also used. Standard construction details and span tables for rafters and ceiling joists can be found in the *International One- and Two-Family Dwelling Code* (ICC, 1998). These tables generally provide allowable horizontal rafter span with disregard to any difference that roof slope may have on axial and bending loads experienced in the rafters. This approach is generally considered as standard practice. Example 5.9 demonstrates two design approaches for a simply-supported, sloped rafter as illustrated in Figure 5.8.

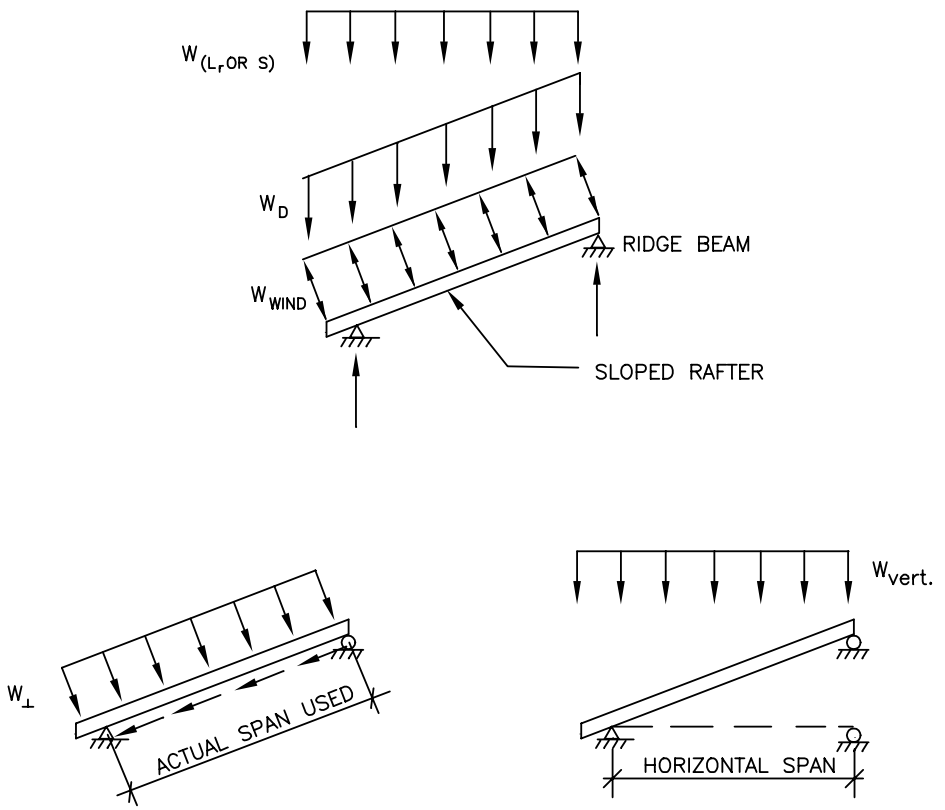
Structural ridge beams are designed to support roof rafters at the ridge when there are no ceiling joists or cross-ties to resist the outward thrust of rafters that would otherwise occur. A repetitive member factor, C_r, is applicable if the ridge beam is composed of two or more members (see Table 5.4). It should also be noted that any additional roof system benefit, such as the folded plate action of the roof sheathing diaphragm, goes ignored in its structural contribution to the ridge beam, particularly for steep-sloped roofs. Example 5.10 demonstrates the design approach for ridge beams.

Roofs with hips and valleys are constructed with rafters framed into a hip or valley rafter as appropriate and, in practice, are typically one to two sizes larger than the rafters they support, e.g., 2x8 or 2x10 hip for 2x6 rafters. While hip and valley rafters experience a unique tributary load pattern or area, they are generally designed much like ridge beams. The folded plate effect of the roof sheathing diaphragm provides support to a hip or valley rafter in a manner similar to that discussed for ridge beams. However, beneficial system effect generally goes ignored because of the lack of definitive technical guidance. Nonetheless, the use of design judgment should not be ruled out. Example 5.11 demonstrates the design of a hip rafter.

5.6.3 Roof Trusses

Roof trusses incorporate rafters (top chords) and ceiling joists (bottom chords) into a structural frame fabricated from 2-inch-thick dimension lumber, usually 2x4s or 2x6s. A combination of web members are positioned between the top and bottom chords, usually in triangular arrangements that form a rigid framework. Many different truss configurations are possible, including open trusses for attic rooms and cathedral or scissor trusses with sloped top and bottom chords. The wood truss members are connected by metal truss plates punched with barbs (i.e., teeth) that are pressed into the truss members. Roof trusses are able to span the entire width of a home without interior support walls, allowing complete freedom in partitioning interior living space. The *Metal Plate Connected Wood Truss Handbook* contains span tables for typical truss designs (WTCA, 1997).

FIGURE 5.8 Design Methods and Assumptions for a Sloped Roof Rafter



Roof truss manufacturers normally provide the required engineering design based on the loading conditions specified by the building designer. The building designer is responsible for providing the following items to the truss manufacturer for design:

- design loads;
- truss profile;
- support locations; and
- any special requirements.

The building designer should also provide for permanent bracing of the truss system at locations designated by the truss designer. In general, such bracing

may involve vertical cross-bracing, runners on the bottom chord, and bracing of certain web members. In typical light-frame residential roof construction, properly attached roof sheathing provides adequate overall bracing of the roof truss system and ceiling finishes normally provide lateral support to the bottom chord of the truss. The only exception is long web members that may experience buckling from excessive compressive loads. Gable endwall bracing is discussed separately in Section 5.6.6 as it pertains to the role of the roof system in supporting the walls against lateral loads, particularly those produced by wind. For more information and details on permanent bracing of trusses, refer to *Commentary for Permanent Bracing of Metal Plate Connected Wood Trusses* (WTCA, 1999). Temporary bracing during construction is usually the responsibility of the contractor and is important for worker safety. For additional guidance on temporary bracing, consult the *Metal Plate Connected Wood Truss Handbook* pages 14-1 through 15-12 and Appendix L (WTCA, 1997). For additional guidance on roles and responsibilities, refer to *Standard Practice for Metal Plate Connected Wood Truss Design Responsibilities* (WTCA, 1995).

The National Design Standard for Metal Plate Connected Wood Truss Construction (ANSI/TPI 1-95) governs the design of trusses. Available from the Truss Plate Institute (TPI, 1995a and b), ANSI/TPI 1-95 includes the structural design procedure as well as requirements for truss installation and bracing and standards for the manufacture of metal plate connectors. A computer program, PPSA, is also available for a detailed finite element analysis (Triche and Suddarth, 1993). Truss plate manufacturers and truss fabricators generally have proprietary computerized design software based on ANSI/TPI 1-95, with modifications tailored to their particular truss-plate characteristics.

The designer should note that cracking and separation of ceiling finishes may occur at joints between the walls and ceiling of roofs. In the unfavorable condition of high attic humidity, the top chord of a truss may expand while the lower roof members, typically buried under attic insulation, may not be similarly affected. Thus, a truss may bow upward slightly. Other factors that commonly cause interior finish cracking are not in any way associated with the roof truss, including shrinkage of floor framing members, foundation settlement, or heavy loading of a long-span floor resulting in excessive deflection that may “pull” a partition wall downward from its attachment at the ceiling. To reduce the potential for cracking of ceiling finishes at partition wall intersections, 2x wood blocking should be installed at the top of partition wall plates as a backer for the ceiling finish material (i.e., gypsum board). Ceiling drywall should not be fastened to the blocking or to the truss bottom chord within 16 to 24 inches of the partition. Proprietary clips are available for use in place of wood blocking and resilient metal “hat” channels may also be used to attach the ceiling finish to the roof framing. Details that show how to minimize partition-ceiling separation problems can be found on the WTCA website at (www.woodtruss.com) or by contacting WTCA to obtain a “Partition Separation” brochure.

Trusses are also frequently used for floor construction to obtain long spans and to allow for the placement of mechanical systems (i.e., ductwork and sanitary drains) in the floor cavity. In addition, trusses have been used to provide a complete house frame (NAHBRC, 1982). One efficient use of a roof truss is as a structural truss for the gable end above a garage opening to effectively eliminate the need for a garage door header. For other efficient framing design concepts and

ideas, refer to *Cost Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994).

5.6.4 Roof Sheathing

Roof sheathing thickness is typically governed by the spacing of roof framing members and live or snow loads. Sheathing is normally in accordance with prescriptive sheathing span rating tables published in a building code or made available by manufacturers. If the limit of the prescriptive tables is exceeded, the designer may need to perform calculations; however, such calculations are rarely necessary in residential construction. The process of selecting rated roof sheathing is similar to that for floor sheathing in Example 5.5.

The fasteners used to attach sheathing to roof rafters are primarily nails. The most popular nail types are sinker, box, and common, of which all have different characteristics that affect structural properties (refer to Chapter 7). Proprietary power-driven fasteners (i.e., pneumatic nails and staples) are also used extensively. The building codes and APA tables recommend a fastener schedule for connecting sheathing to roof rafters. Generally, nails are placed at a minimum 6 inches on center at edges and 12 inches on center at intermediate supports. A 6-inch fastener spacing should also be used at the gable-end framing to help brace the gable-end. Nail size is typically 8d, particularly since thinner power driven nails are most commonly used. Roof sheathing is commonly 7/16- to 5/8-inch-thick on residential roofs. Note that in some cases shear loads in the roof diaphragm resulting from lateral loads (i.e., wind and earthquake) may require a more stringent fastening schedule; refer to Chapter 6 for a discussion of fastening schedules for lateral load design. More importantly, large suction pressures on roof sheathing in high wind areas (see Chapter 3) will require a larger fastener and/or closer spacing. In hurricane-prone regions, it is common to require an 8d deformed shank nail with a 6 inch on center spacing at all framing connections. At the gable end truss or rafter, a 4 inch spacing is common.

5.6.5 Roof Overhangs

Overhangs are projections of the roof system beyond the exterior wall line at either the eave or the rake (the sloped gable end). Overhangs protect walls from rain and shade windows from direct sun. When a roof is framed with wood trusses, an eave overhang is typically constructed by extending the top chord beyond the exterior wall. When a roof is framed with rafters, the eave overhang is constructed by using rafters that extend beyond the exterior wall. The rafters are cut with a “bird-mouth” to conform to the bearing support. Gable end overhangs are usually framed by using a ladder panel that cantilevers over the gable end for either stick-framed or truss roofs. Refer to Figure 5.9 for illustrations of various overhang constructions.

A study completed in 1978 by the Southern Forest Experiment Station for the U.S. Department of Housing and Urban Development found that the protection afforded by overhangs extends the life of the wall below, particularly if the wall is constructed of wood materials (HUD, 1978). Entitled the *Prevention and Control of Decay in Homes*, the report correlates the climate index of a

geographic area with a suggested overhang width and recommends highly conservative widths. As a reasonable guideline (given that in many cases no overhang is provided), protective overhang widths should be 12 to 24 inches in moist, humid climates and more if practicable. A reasonable rule-of-thumb to apply is to provide a minimum of 12 inches of overhang width for each story of protected wall below. However, overhang width can significantly increase wind uplift loads on a roof, particularly in high wind regions. The detailing of overhang framing connections (particularly at the rake overhang on a gable end) is a critical consideration in hurricane-prone regions. Often, standard metal clips or straps provide adequate connection. The need for special rake overhang design detailing depends on the length of the overhang, the design wind load condition, and the framing technique that supports the overhang (i.e., 2x outriggers versus cantilevered roof sheathing supporting ladder overhang framing).

5.6.6 Gable-End Wall Bracing

Roof framing provides lateral support to the top of the walls where trusses and rafters are attached to the wall top plate. Likewise, floor framing provides lateral support to the top and bottom of walls, including the top of foundation walls. At a gable end, however, the top of the wall is not directly connected to roof framing members; instead, it is attached to the bottom of a gable-end truss and lateral support at the top of the wall is provided by the ceiling diaphragm. In higher-wind regions, the joint may become a “hinge” if the ceiling diaphragm becomes overloaded. Accordingly, it is common practice to brace the top of the end wall (or bottom of the gable end roof framing) with 2x4 or 2x6 framing members that slope upward to the roof diaphragm to attach to a blocking or a ridge “beam” as shown in Figure 5.9. Alternatively, braces may be laid flatwise on ceiling joists or truss bottom chords and angled to the walls that are perpendicular to the gable-end wall. Given that braces must transfer inward and outward forces resulting from positive wind pressure or suction on the gable-end wall, they are commonly attached to the top of the gable-end wall with straps to transfer tension forces that may develop in hurricanes and other extreme wind conditions. The need for and special detailing of gable-end wall braces depends on the height and area of the gable end (i.e., tributary area) and the design wind load. The gable endwall can also be braced by the use of a wood structural panel attached to the gable end framing and the ceiling framing members.

As an alternative to the above strategy, the gable-end wall may be framed with continuous studs that extend to the roof sheathing at the gable end (i.e., balloon-framed). If the gable-end wall encloses a two-story room–such as a room with a cathedral ceiling, it is especially important that the studs extend to the roof sheathing; otherwise, a hinge may develop in the wall and cause cracking of wall finishes (even in a moderate wind) and could easily precipitate failure of the wall in an extreme wind. Depending on wall height, stud size, stud spacing, and the design wind load condition, taller, full-height studs may need to be increased in size to meet deflection or bending capacity requirements. Some designer judgment should be exercised in this framing application with respect to the application of deflection criteria. The system deflection adjustment factors of

FIGURE 5.9 Typical Roof Overhang Construction

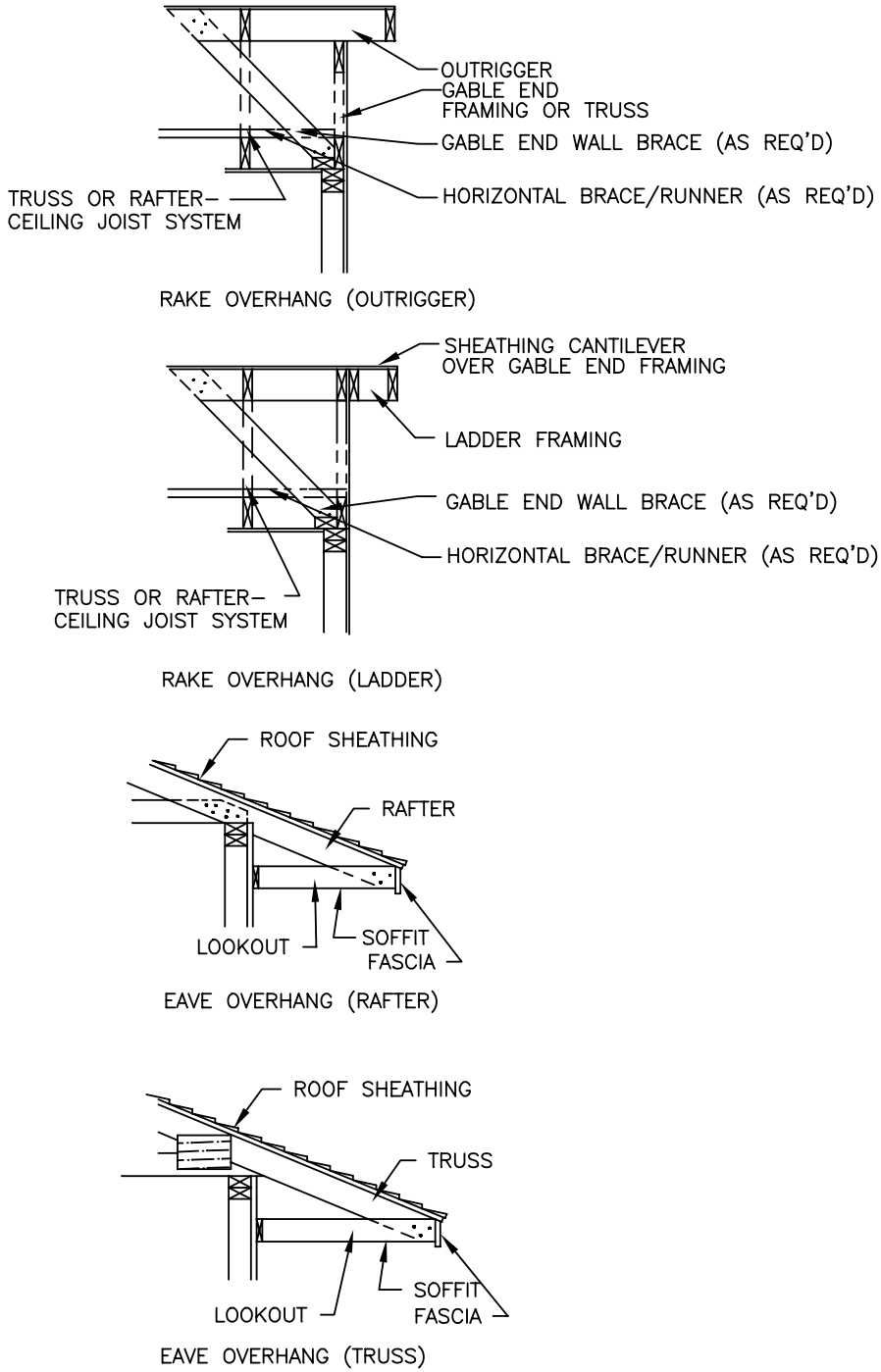


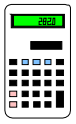
Table 5.6 may assist in dealing with the need to meet a reasonable serviceability limit for deflection (see Section 5.3.2).

Finally, as an alternative that avoids the gable-end wall bracing problem, a hip roof may be used. The hip shape is inherently more resistant to wind damage in hurricane-prone wind environments (see Chapter 1) and braces the end walls against lateral wind loads by direct attachment to rafters.

5.7 Design Examples

In this section, a number of design examples illustrate the design of various elements discussed in this chapter. The examples are intended to also provide practical advice. Therefore, the examples are embellished with numerous notes and recommendations to improve the practicality and function of various possible design solutions. They are also intended to promote the designer's creativity in arriving at the best possible solution for a particular application

EXAMPLE 5.1 Typical Simple Span Floor Joist Design



Given

Live load (L)	=	30 psf (bedroom area)
Dead load (D)	=	10 psf
Trial joist spacing	=	16 on center
Trial joist size	=	2x8
Trial joist species and grade	=	Hem-Fir, No. 1 (S-dry, 19% MC)

Find Maximum span for specified joist member.

Solution

1. Determine tabulated design values by using NDS-S (Tables 4A and 1B)

F_b	=	975 psi	I_{xx}	=	47.63 in^4
F_v	=	75 psi	S_{xx}	=	13.14 in^3
$F_{c\perp}$	=	405 psi	b	=	1.5 in
E	=	1,500,000 psi	d	=	7.25 in

2. Lumber property adjustments and adjusted design values (Section 5.2.4 and NDS•2.3)

C_D = 1.0 (Section 5.2.4.1)
C_r = 1.15 (Table 5.4)
C_F = 1.2 (NDS-S Table 4A adjustment factors)
C_H = 2.0 (Section 5.2.4.3)
C_L = 1.0 (NDS•3.3.3, continuous lateral support)
C_b = 1.0 (NDS•2.3.10)

$F_b' = F_bC_rC_FC_DC_L = 975\ (1.15)(1.2)(1.0)(1.0) = 1{,}345$ psi
$F_v' = F_vC_HC_D = 75\ (2)(1.0) = 150$ psi
$F_{c\perp}' = F_{c\perp}C_b = 405\ (1.0) = 405$ psi
$E' = E = 1{,}500{,}000$ psi

3. Calculate the applied load

W = (joist spacing)(D+L) = (16 in)(1 ft/12 in)(40 psf) = 53.3 plf

4. Determine maximum clear span based on bending capacity

$$M_{max} = \frac{w\ell^2}{8} = \frac{(53.3\ \text{plf})(\ell^2)}{8} = 6.66\,\ell^2$$

$$f_b = \frac{M}{S} = \frac{(6.66\ell^2)(12\ \text{in/ft})}{13.14\ \text{in}^3} = 6.08\,\ell^2$$

$f_b \le F_b'$
$6.08\ \ell^2 \le 1{,}345$ psi
$\ell^2 = 221$
$\ell = 14.9$ ft = 14 ft-11 in (maximum clear span due to bending stress)

5. Determine maximum clear span based on horizontal shear capacity

$$V_{max} = \frac{w\ell}{2} = \frac{(53.3\ \text{plf})(\ell)}{2} = 26.7\ \ell$$

$$f_v = \frac{3V}{2A} = \frac{3}{2}\left(\frac{26.7\ \ell}{(1.5\ \text{in})(7.25\ \text{in})}\right) = 3.7\ \ell$$

$f_v \leq F_v'$

$3.7\ell \leq 150$ psi

ℓ = 40.5 ft = 40 ft-6 in (maximum clear span due to horizontal shear stress)

6. Determine maximum clear span based on bearing capacity

Bearing length = (3.5-in top plate width) - (1.5-in rim joist width) = 2 in

$$f_{c\perp} = \frac{\frac{1}{2}w\ell}{A_b} = \frac{\frac{1}{2}(53.3\ \text{plf})(\ell)}{(2\ \text{in})(1.5\ \text{in})} = 8.9\ \ell$$

$f_{c\perp} < F_{c\perp}'$

$8.9\ell \leq 405$ psi

ℓ = 45.5 ft = 45 ft-6 in (maximum clear span due to bearing stress)

7. Consider maximum clear span based on deflection criteria (Section 5.3.2)

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(40\ \text{plf})^*(\ell)^4\ (1{,}728\ \text{in}^3/\text{ft}^3)}{384(1{,}500{,}000\ \text{psi})(47.63\ \text{in}^4)} = 1.26 \times 10^{-5}\ell^4$$

*applied live load of 30 psf only

$$\rho_{all} = \frac{\ell}{360}(12\ \text{in/ft}) = 0.033\ \ell$$

$\rho_{max} \leq \rho_{all}$

$1.26 \times 10^{-5}\ \ell^4 \leq 0.033\ \ell$

$\ell^3 = 2{,}619$

ℓ = 13.8 ft = 13 ft-10 in (recommended clear span limit due to deflection criteria)

8. Consider floor vibration (Section 5.3.2)

The serviceability deflection check was based on the design floor live load for bedroom areas of 30 psf. The vibration control recommended in Section 5.3.2 recommends using a 40 psf design floor live load with the ℓ/360 deflection limit. Given that the span will not be greater than 15 feet, it is not necessary to use the absolute deflection limit of 0.5 inch.

w = (16 in)(1 ft/12 in)(40 psf) = 53.3 plf

$$\rho_{all} = (\ell/360)(12\ \text{in/ft}) = 0.033\ \ell$$

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(53.3\ \text{plf})^*(\ell^4)\ (1{,}728\ \text{in}^3/\text{ft}^3)}{384(1.5 \times 10^6\ \text{psi})(47.63\ \text{in}^4)} = 1.7 \times 10^{-5}\ell^4$$

*applied live load of 40 psf only

$\rho_{max} \leq \rho_{all}$

$1.7 \times 10^{-5}\ \ell^4 \leq 0.033\ \ell$

$\ell^3 = 1{,}941$

ℓ = 12.5 ft = 12 ft-6 in (recommended clear span limit due to vibration)

Conclusion

The serviceability limit states used for deflection and floor vibration limit the maximum span. The deflection limited span is 13 ft-10 in and the vibration limited span is 12 ft-6 in. Span selection based on deflection or vibration is an issue of designer judgment. The maximum span limited by the structural safety checks was 14 ft-11 in due to bending. Therefore, the serviceability limit will provide a notable safety margin above that required. Thus, No. 2 grade lumber should be considered for economy in that it will have only a small effect on the serviceability limits. Conversely, if floor stiffness is not an expected issue with the owner or occupant, the span may be increased beyond the serviceability limits if needed to "make it work." Many serviceable homes have been built with 2x8 floor joists spanning as much as 15 feet; however, if occupants have a low tolerance for floor vibration, a lesser span should be considered.

For instructional reasons, shrinkage across the depth of the floor joist or floor system may be estimated as follows based on the equations in Section 5.3.2:

d_1 = 7.25 in M_1 = 19% maximum (S-dry lumber)
d_2 = ? M_2 = 10% (estimated equilibrium MC)

$$d_2 = d_1 \left(\frac{1 - \frac{a - 0.2M_2}{100}}{1 - \frac{a - 0.2M_1}{100}} \right) = 7.25 \text{ in} \left(\frac{1 - \frac{6.031 - 0.2(10)}{100}}{1 - \frac{6.031 - 0.2(19)}{100}} \right) = 7.1 \text{ in}$$

Shrinkage ≅ 7.25 ft-7.08 in = 0.15 in (almost 3/16 in)

In a typical wood-framed house, shrinkage should not be a problem, provided that it is uniform throughout the floor system. In multistory platform frame construction, the same amount of shrinkage across each floor can add up to become a problem, and mechanical systems and structural details should allow for such movement. Kiln-dried lumber may be specified to limit shrinkage and building movement after construction.

EXAMPLE 5.2 *Simple Span Floor Joist Design (Optimize Lumber)*

Given

Live load (L) = 40 psf
Dead load (D) = 10 psf
Clear span = 14 ft-2 in
Joist size = 2x10

Find Optimum lumber species and grade

Solution

1. Calculate the applied load

W = (joist spacing)(D+L) = (2 ft)(40 psf +10 psf) = 100 plf

2. Determine bending stress

$$M_{max} = \frac{w\ell^2}{8} = \frac{(100\,plf)(14.17\,ft)^2}{8} = 2{,}510\ ft\text{-}lb$$

$$F_b = \frac{M}{S} = \frac{(2{,}510\ ft-lb)(12\ in/ft)}{21.39\ in^3} = 1{,}408\ psi$$

3. Determine horizontal shear stress

$$V_{max} = \frac{w\ell}{2} = \frac{(100\ plf)(14.17\ ft)}{2} = 709\ lb$$

$$f_v = \frac{3V}{2A} = \frac{3(709\ lb)}{2(1.5\ in)(9.25\ in)} = 77\ psi$$

4. Determine bearing stress:

$$R_1 = R_2 = V_{max} = 709\ lb$$

$$f_{c\perp} = \frac{R}{A_b} = \frac{709\ lb}{(2\ in)(1.5\ in)} = 236\ psi$$

Wall and roof loads, if any, are carried through rim/band joist

5. Determine minimum modulus of elasticity due to selected deflection criteria

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(80\ plf)*(14.17\ ft)^4\,(1{,}728\,in^3/ft^3)}{384E\ (98.93\ in^4)} = 733{,}540/E$$

*includes live load of 40 psf only

$$\rho_{all} \le \ell/360$$

$$\rho_{max} \le \rho_{all}$$

$$\frac{733{,}540}{E} = \frac{(14.17\ ft)(12\ in/ft)}{360}$$

$$E_{min} = 1.55 \times 10^6\ psi$$

6. Determine minimum modulus of elasticity due to vibration

The span required is not greater than 15 feet and the $\ell/360$ deflection check uses a 40 psf floor live load. Therefore, the deflection check is assumed to provide adequate vibration control.

7. Determine minimum required unadjusted properties by using NDS tabulated lumber data

Bending

$$f_b \leq F_b'$$
$$F_b' = F_b C_r C_F C_D$$
$$F_{bmin} = \frac{f_b}{C_r C_F C_D} = \frac{1{,}408 \text{ psi}}{(1.15)(1.1)(1.0)} = 1{,}113 \text{ psi}$$

Horizontal shear

$$f_v \leq F_v'$$
$$F_v' = F_v C_H C_D$$
$$F_{vmin} = \frac{f_v}{C_H C_D} = \frac{77 \text{ psi}}{(2)(1.0)} = 39 \text{ psi}$$

Bearing

$$f_{c\perp} \leq F_{c\perp}' \quad \text{(assume minimum 2-in bearing)}$$
$$F_{c\perp} = F_{c\perp} C_b$$
$$F_{c\perp min} = \frac{f_{c\perp}}{(1.0)} = 236 \text{ psi}$$

Minimum unadjusted tabulated properties required

F_b =	1,113 psi	$F_{c\perp}$ =	236 psi
F_v =	39 psi	E =	1.55×10^6 psi

8. Select optimum lumber grade considering local availability and price by using NDS-S Table 4A or 4B data

Minimum No. 2 grade lumber is recommended for floor joists because of factors related to lumber quality such as potential warping and straightness that may affect constructability and create call-backs.

Considering 2x10 Douglas Fir-Larch, the grade below (No. 1 and Btr) was selected to meet the required properties.

F_b	=	1,200 psi	>	1,113 psi	OK
F_v	=	95 psi	>	39 psi	OK
$F_{c\perp}$	=	625 psi	>	236 psi	OK
E	=	1.8×10^6 psi	>	1.55×10^6 psi	OK

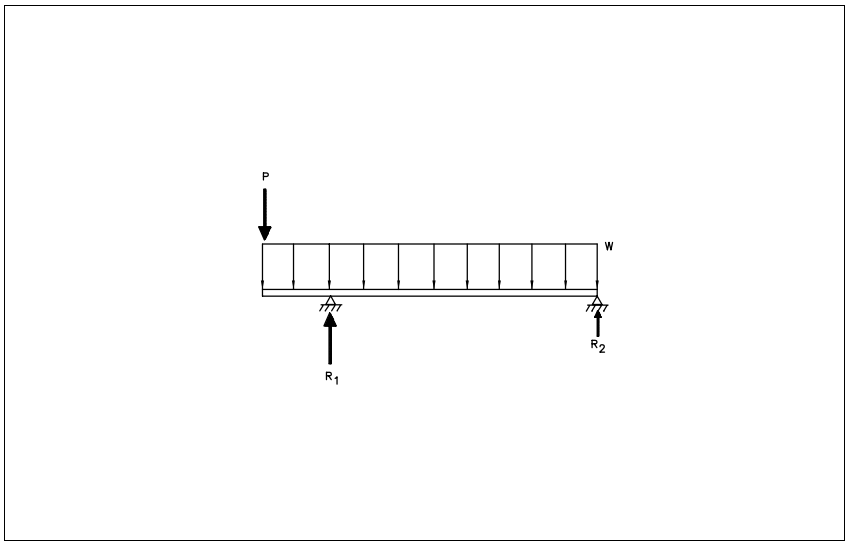
Conclusion

Many other species and grades should be considered depending on local availability and cost. Also, the No. 1 and higher grades are generally considered as “premium” lumber. A more economical design may be possible by using a closer joist spacing to allow for a lower grade (i.e, 19.2 inches on center or 16 inches on center). Also, a lower grade 2x12 should be considered or, perhaps, engineered wood I-joists.

EXAMPLE 5.3 Cantilevered Floor Joist

Given

Joist spacing = 16 in on center
Joist size = 2x10
Bearing length = 3-1/2 in
Species = Douglas Fir-Larch, No.1 Grade
Loads on cantilever joist (see Chapter 3)
Floor live load (L) = 40 psf
Floor dead load (D) = 10 psf
Loads for concentrated load at end of cantilever (see Chapter 3)
Roof snow load (S) = 11 psf (15 psf ground snow load and 7:12 roof pitch)
Roof dead load (D) = 12 psf
Wall dead load (D) = 8 psf
Roof span = 28 ft (clear span plus 1 ft overhang)
Wall height = 8 ft



Cantilever Joist Load Diagram

Find

Determine the maximum cantilever span for the specified floor joist based on these load combinations (Chapter 3, Table 3.1):

$D + L + 0.3 (S \text{ or } L_r)$
$D + (S \text{ or } L_r) + 0.3L$

The analysis does not consider wind uplift that may control connections in high-wind areas, but not necessarily the cantilever joist selection.

Deflection at the end of the cantilever should be based on a limit appropriate to the given application. The application differs from a normal concern with mid-span deflection; experience indicates that deflection limits can be safely and serviceably relaxed in the present application. A deflection limit of $\ell/120$ inches at the end of cantilever is recommended, particularly when the partial composite action of the sheathing is neglected in determining the moment of inertia, I, for the deflection analysis.

Solution

1. Determine tabulated design values for species and grade from the NDS-S

F_b	=	1000 psi	S	=	21.39 in^3
F_v	=	95 psi	I	=	98.93 in^4
$F_{c\perp}$	=	625 psi	b	=	1.5 in
E	=	$1.7x10^6$ psi	d	=	9.25 in

2. Determine lumber property adjustments (see Section 5.2.4)

C_r	=	1.15	C_F	=	1.1
C_H	=	2.0	C_D	=	1.25 (includes snow)
C_b^*	=	1.11	C_L	=	1.0 (continuous lateral support)**

*Joist bearing not at end of member (see NDS•2.3.10)

**The bottom (compression edge) of the cantilever is assuemd to be laterally braced with wood structural panel sheathing or equivalent. If not, the value of CL is dependent on the slenderness ratio (see NDS•3.3.3).

$F_b' = F_bC_rC_FC_DC_L = (1000 \text{ psi})(1.15)(1.1)(1.25)(1.0) = 1{,}581 \text{ psi}$
$F_v' = F_vC_HC_D = (95)(2)(1.25) = 238 \text{ psi}$
$F_{c\perp}' = F_{c\perp}C_b = 625\ (1.11) = 694 \text{ psi}$
$E' = E = 1.7x10^6 \text{ psi}$

3. Determine design loads on cantilever joist

The following load combinations (based on Chapter 3, Table 3.1) will be investigated for several load cases that may govern different safety or serviceability checks

Case I: D+S - Cantilever Deflection Check
P = wall and roof load (lb) at end of cantilever = f(D+S)
w = uniform load (plf) on joist = f(D only)

Case II: D+L - Deflection at Interior Span
P = f(D only)
w = f(D+L)

Case III: D+S+0.3L or D+L+0.3S - Bending and Horizontal Shear at Exterior Bearing Support
a. P = f(D+S)
w = f(D + 0.3L)

b. P = f(D+0.3S)
w = f(D+L)

The following values of P and W are determined by using the nominal design loads, roof span, wall height, and joist spacing given above

	Case I	Case II	Case IIIa	Case IIIb
P =	544 lb	325 lb	544 lb	390 lb
W =	13.3 plf	66.5 plf	29.3 plf	66.5 lb

Inspection of these loading conditions confirms that Case I controls deflection at the end of the cantilever, Case II controls deflection in the interior span, and either Case IIIa or IIIb controls the structural safety checks (i.e., bending, horizontal shear, and bearing).

Since the cantilever span, *X*, is unknown at this point, it is not possible to determine structural actions in the joist (i.e., shear and moment) by using traditional engineering mechanics and free-body diagrams. However, the beam equations could be solved and a solution for *X* iterated for all required structural safety and serviceability checks (by computer). Therefore, a trial value for *X* is determined in the next step. If an off-the-shelf computer program is used, verify its method of evaluating the above load cases.

4. Determine a trial cantilever span based on a deflection limit of $\ell/120$ and load Case I.

Use a 2 ft-10 in cantilever span (calculations not shown - see beam equations in Appendix A).

5. Determine the maximum bending moment and shear for the three load cases governing the structural safety design checks by using the trail cantilever span:

The following is determined by using free-body diagrams and shear and moment diagrams (or beam equations, see Appendix A)

	Case II	Case IIIa	Case IIIb
R_1	1,008 lb	938 lb	**1,088 lb**
R_2	301 lb	40 lb	286 lb
V_{max}*	511 lb	**626 lb**	576 lb
M_{max}	1,170 ft-lb	**1,638 ft-lb**	1,352 lb

**NDS•3.4.3 allows loads within a distance of the member depth, d, from the bearing support to be ignored in the calculation of shear V when checking horizontal shear stress. However, this portion of the load must be included in an analysis of the bending moment. It would reduce the value of V_{max} as calculated above by using beam equations by approximately 100 pounds in Case II and Case IIIb and about 44 pounds in Case IIIa by eliminating the uniform load, w, within a distance, d, from the exterior bearing support.*

6. Determine design bending moment capacity of the given joist and verify adequacy

$$F_b' \geq f_b = \frac{M_{all}}{S}$$

$$M_{all} = F_b'S = (1{,}581 \text{ psi})(21.4 \text{ in}^3)(1 \text{ ft}/12 \text{ in})$$
$$= 2{,}819 \text{ ft-lb}$$
$$M_{all} > M_{max} = 1{,}638 \text{ ft-lb} \quad \text{OK}$$

7. Determine design shear capacity of the given joist and verify adequacy:

$$F_v = \frac{3V_{all}}{2A} \text{ and } F_v \geq F_v'$$

$$V_{all} = \frac{2AF_v'}{3} = \frac{2(1.5 \text{ in})(9.25 \text{ in})(238 \text{ psi})}{3}$$
$$= 2{,}202 \text{ lbs}$$
$$V_{all} > V_{max} = 626 \text{ lbs} \quad \text{OK}$$

8. Check bearing stress

$$f_{c\perp} = \frac{R_{max}}{A_b} = \frac{1{,}088 \text{ lb}}{(1.5 \text{ in})(3.5 \text{ in})}$$

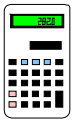
$$= 207 \text{ psi}$$

$$F_{c\perp}' = 694 \text{ psi} > 207 \text{ psi} \quad \text{OK}$$

Conclusion

A cantilever span of 2 ft-10 in (2.8 feet) is structurally adequate. The span is controlled by the selected deflection limit (i.e., serviceability) which illustrates the significance of using judgment when establishing and evaluating serviceability criteria. Allowance for a 2-foot cantilever is a common field practice in standard simple span joist tables for conventional residential construction. A check regarding interior span deflection of the joist using load Case II may be appropriate if floor vibration is a concern. However, unacceptable vibration is unlikely given that the span is only 12 feet. Also, Douglas-Fir, Larch, No. 1 Grade, is considered premium framing lumber and No. 2 Grade member should be evaluated, particularly if only a 2-foot cantilever is required.

EXAMPLE 5.4 Built-Up Floor Girder Design



Given

Loads

Floor live load	=	40 psf
Floor dead load	=	10 psf
Required girder span (support column spacing)	=	14 ft
Joist span (both sides of girder)	=	12 ft
Species	=	Southern Pine, No. 1
Maximum girder depth	=	12

Find Minimum number of 2x10s or 2x12s required for the built-up girder.

Solution

1. Calculate the design load

W = (Trib. floor joist span)(D + L) = (12 ft)(40 psf + 10 psf) = 600 plf

2. Determine tabulated design values (NDS-S Table 4B)

F_b = 1250 psi $F_{C\perp}$ = 565 psi
F_v = 90 psi E = 1.7×10^6 psi

3. Lumber property adjustments (Section 5.2.4):

C_r = 1.2 (Table 5.4) C_D = 1.0
C_F = 1.0 C_b = 1.0
C_H = 2.0 C_L = 1.0

(compression flange laterally braced by connection of floor joists to top or side of girder)

$F_b' = F_b C_D C_r C_F C_L$ = 1,250 psi (1.0)(1.2)(1)(1) = 1,500 psi
$F_V' = F_l C_D C_H$ = 90 psi (1.25)(2.0) = 225 psi
$F_{c\perp}' = F_{c\perp} C_b$ = 565 psi (1) = 565 psi
E' = E = 1.7×10^6 psi

4. Determine number of members required due to bending

$$M_{max} = \frac{w\ell^2}{8} = \frac{(600\text{plf})(14\text{ ft})^2}{8} = 14{,}700 \text{ ft-lb}$$

$$f_b = \frac{M}{S} = \frac{(14{,}700 \text{ ft} - \text{lb})(12 \text{ in/ft})}{S} = \frac{176{,}400}{S}$$

$$f_b \le F_b'$$

$$\frac{176{,}400}{S} \le 1{,}500 \text{ psi}$$

$$S_x = 118 \text{ in}^3$$

Using Table 1B in NDS-S

5 2x10s S = 5(21.39) = 107 < 118 (marginal, but 5 too thick)
4 2x12s S = 4 (31.64) = 127 > 118 (OK)

5. Determine number of members required due to horizontal shear

$$V_{max} = \frac{w\ell}{2} = \frac{600 \text{ plf } (14 \text{ ft})}{2} = 4{,}200 \text{ lb}$$

$$f_v = \frac{3V}{2A} = \frac{3}{2}\left(\frac{4200}{A}\right) = 6{,}300 \text{ lb}/A$$

$$f_v \leq F_v'$$

$$\frac{6{,}300 \text{ lb}}{A} \leq 225 \text{ psi}$$

$$A = 28 \text{ in}^2$$

2 2x12s A = 33.8 > 28 OK

2 2x10s A = 27.8 ≈ 28 OK

6. Determine required bearing length using 4 2x12s

$$R_1 = R_2 = V_{max} = 4{,}200 \text{ lb}$$

$$f_{c\perp} = \frac{R}{A_b} = \frac{4{,}200 \text{ lb}}{(6 \text{ in})(\ell_b)} = \frac{700}{\ell_b}$$

$$f_{c1} \leq F_{c\perp}'$$

$$\frac{700}{\ell_b} \leq 565 \text{ psi}$$

$$\ell_b = 1.24 \text{ in} \quad \text{(OK)}$$

7. Determine member size due to deflection

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(480 \text{ plf})*(14 \text{ ft})^4 (1{,}728 \text{ in}^3/\text{ft}^3)}{384\,EI} = \frac{4.15 \times 10^8}{EI}$$

*includes 40 psf live load only

$$\rho_{all} \leq \ell/360 = 14 \text{ ft}(12 \text{ in/ft})/360 = 0.47 \text{ in}$$

$$\rho_{max} \leq \rho_{all}$$

$$\frac{4.15 \times 10^8}{EI} = 0.47 \text{ in}$$

$$EI = 8.8 \times 10^8$$

$$(1.7 \times 10^6)(I) = 8.8 \times 10^8$$

$$I = 519 \text{ in}^4$$

3 2x12s I = 534 > 519 okay

8. Check girder for floor system vibration control (see Section 5.3.2)

Girder span, $\ell_1 = 14$ ft
Joist span, $\ell_2 = 12$ ft
$\ell_{TOTAL} = 26$ ft > 20 ft

Therefore, check girder using $\ell/480$ or $\ell/600$ to stiffen floor system

Try $\ell/480$

$$\rho_{max} = \frac{4.15 \times 10^8}{EI} \text{ (as before)}$$

$$\rho_{all} = \ell/480 = \frac{14 \text{ ft} (12 \text{ in / ft})}{480} = 0.35 \text{ in}$$

$$\rho_{max} \leq \rho_{all}$$

$$\frac{4.15 \times 10^8}{EI} = 0.35 \text{ in}$$

$$EI = 1.2 \times 10^9$$

$$I = \frac{1.2 \times 10^9}{1.7 \times 10^6} = 706 \text{ in}^4$$

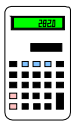
Using Table 1B in NDS, use

4 2x12s $I = 4 (178 \text{ in}^4) = 712 \text{ in}^4 > 706 \text{ in}^4$ OK

Conclusion

The bending stress limits the floor girder design to 4 2x12's (No. 1, SYP). The use of 4 2x12s also provides a "stiff" girder with respect to floor vibration (i.e., deflection limit of $\ell/480$. As a practical alternative, a steel "floor beam" (e.g., W-shape) or an engineered wood beam may also be used, particularly if "clearance" is a concern.

EXAMPLE 5.5 Subfloor Sheathing Design



Given

Joist spacing = 16 in on center
Floor live load = 40 psf
Use APA rated subflooring

Find The required sheathing span rating and thickness with the face grain perpendicular to the joist span.

Determine size and spacing of fasteners.

Solution

Determine sheathing grade and span rating and thickness by using the APA's *Design and Construction Guide for Residential and Commercial* (APA, 1998). From Table 7 in the APA guide, use 7/16-inch-thick (24/16 rating) sheathing or 15/32-inch- to 1/2-inch-thick (32/16 rating) sheathing. The first number in the rating applies to the maximum spacing of framing members for roof applications; the second to floor applications. It is fairly common to up size the sheathing to the next thickness, e.g., 3/4-inch, to provide a stiffer floor surface. Such a decision often depends on the type of floor finish to be used or the desired "feel" of the floor. Similar ratings are also available from other structural panel trademarking organizations and also comply with the PS-2 standard. It is important to ensure that the sheathing is installed with the long dimension (i.e., face grain) perpendicular to the floor framing; otherwise, the rating does not apply. For wall applications, panel orientation is not an issue.

Use 6d common nails for 7/16-inch-thick sheathing or 8d common nails for thicknesses up to 1 inch (see Table 5.7). Nails should be spaced at 6 inches on center along supported panel edges and 12 inches on center along intermediate supports.

Conclusion

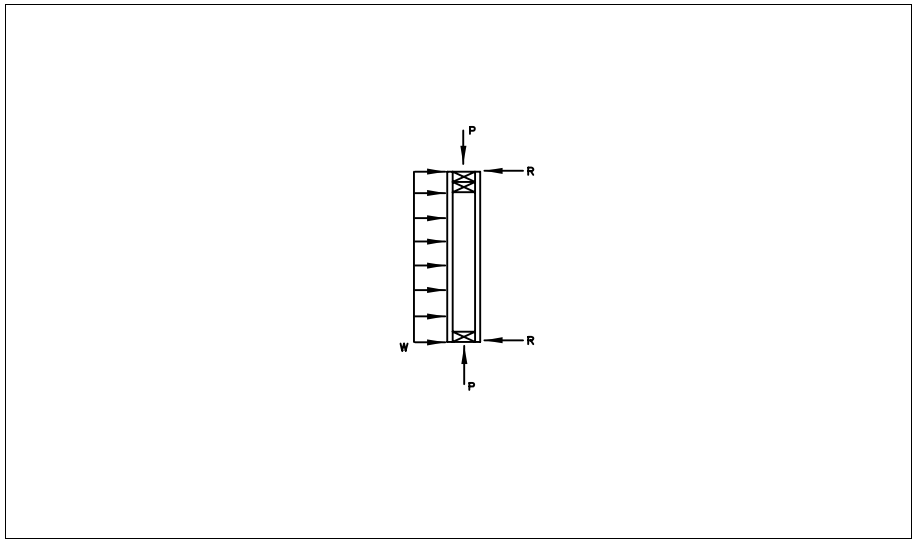
Sheathing design involves matching the proper sheathing rating with the floor framing spacing and live load condition. The process is generally a "cook book" method that follows tables in manufacturer's literature or the applicable building code. Board sheathing and decking are other possible subfloor options that may be designed by using the NDS. Prescriptive tables for these options are also generally available in wood industry publications or in the applicable residential building code.

EXAMPLE 5.6 Exterior Bearing Wall Design

Given

Stud size and spacing	=	2x4 at 24 in on center
Wall height	=	8 ft
Species and grade	=	Spruce-Pine-Fir, Stud Grade
Exterior surface	=	7/16-in-thick OSB
Interior surface	=	1/2-in-thick, gypsum wall board
Wind load (100 mph, gust)	=	16 psf (see Chapter 3, Example 3.2)

Find Vertical load capacity of stud wall system for bending (wind) and axial compression (dead load) and for axial compression only (i.e., dead, live, and snow loads); refer to Chapter 3, Table 3.1, for applicable load combinations.



Wall Loading Diagram

Solution

1. Determine tabulated design values for the stud by using the NDS-S (Table A4)

F_b	=	675 psi	$F_{c\perp}$	=	425 psi
F_t	=	350 psi	F_c	=	725 psi
F_v	=	70 psi	E	=	1.2×10^6 psi

2. Determine lumber property adjustments (see Section 5.2.4)

C_D = 1.6 (wind load combination)
= 1.25 (gravity/snow load combination)
C_r = 1.5 (sheathed wall assembly, Table 5.4)
C_L = 1.0 (continuous lateral bracing)
C_F = 1.05 for F_c
= 1.1 for F_t
= 1.1 for F_b

3. Calculate adjusted tensile capacity

Not applicable to this design. Tension capacity is OK by inspection.

4. Calculate adjusted bending capacity

$F_b' = F_b C_D C_L C_F C_r = (675)(1.6)(1.0)(1.1)(1.5) = 1{,}782$ psi

5. Calculate adjusted compressive capacity (NDS•3.7)

$F_c^* = F_c C_D C_F = (725 \text{ psi})(1.6)(1.05) = 1{,}218$ psi

$E' = E = 1.2 \times 10^6$ psi

$K_{cE} = 0.3$ visually graded lumber

$c = 0.8$ sawn lumber

$$F_{cE} = \frac{K_{cE} E'}{\left(l_e / d\right)^2} = \frac{0.3(1.2 \times 10^6 \text{ psi})}{\left[8 \text{ ft}(12 \text{ in/ft}) / 3.5 \text{ in}\right]^2} = 479 \text{ psi}$$

$$C_p = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$ (column stability factor)

$$= \frac{1+\left(479/1{,}218\right)}{2(0.8)} - \sqrt{\left[\frac{1+\left(479/1{,}218\right)}{2(0.8)}\right]^2 - \frac{479/1{,}218}{0.8}} = 0.35$$

$F_c' = F_c C_D C_F C_P = (725 \text{ psi})(1.6)(1.05)(0.35) = 426$ psi

Axial load only case

Calculations are same as above except use $C_D = 1.25$
$F_c^* = 952$ psi
$C_p = 0.44$
$F_c' = F_c C_D C_F C_P = 725 \text{ psi } (1.25)(1.05)(0.44) = 419$ psi

6. Calculate combined bending and axial compression capacity for wind and gravity load (dead only) by using the combined stress interaction (CSI) equation (NDS•3.9.2):

$$f_b = \frac{M}{S} = \frac{\frac{1}{8} w \ell^2}{S}$$

$$= \frac{\frac{1}{8}(24 \text{ in})(16 \text{ psf})\left[(8 \text{ ft})(12 \text{ in/ft})\right]^2 (1 \text{ ft}/12 \text{ in})}{3.06 \text{ in}^3}$$

$$= 1{,}004 \text{ psi}$$

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_b}{F_b'\left[1 - f_c/F_{cE1}\right]} \le 1.0$$ (CSI equation for bending in strong axis of stud only)

$$\left(\frac{f_c}{426}\right)^2 + \frac{1{,}004}{1{,}782\left(1 - f_c/479\right)} = 1.0$$ (solve CSI equation for f_c)

$f_{c,\,max}$ = 163 psi/stud

$P = f_c A = (163\text{ psi/stud})(1.5\text{ in})(3.5\text{ in}) = 856$ lb/stud

$$w = (856\text{ lb/stud})\left(\frac{1\text{ stud}}{2\text{ ft}}\right) = 428\text{ plf}$$ (uniform dead load at top of wall)

Therefore, the maximum axial (dead) load capacity is 428 plf with the wind load case (i.e., D+W).

7. Determine maximum axial gravity load without bending load

This analysis applies to the D + L+ 0.3(S or L_r) and D + (S or L_r) + 0.3L load combinations (see Table 3.1, Chapter 3).

Using F_c' determined in Step 5 (axial load only case), determine the stud capacity acting as a column with continuous lateral support in the weak-axis buckling direction.

$F_c \leq F_c'$

$$\frac{P}{A} \leq 419\text{ psi}$$

$P_{max} = (419\text{ psi})(1.5\text{ in})(3.5\text{ in}) = 2{,}200$ lbs/stud

Maximum axial load capacity (without simultaneous bending load) is 2,200 lbs/stud or 1,100 lbs/lf of wall.

8. Check bearing capacity of wall plate

Not a capacity limit state. ($F_{c\perp}$ is based on deformation limit state, not actual bearing capacity.) OK by inspection.

Conclusion

The axial and bending load capacity of the example wall is ample for most residential design conditions. Thus, in most cases, use of the prescriptive stud tables found in residential building codes may save time. Only in very tall walls (i.e., greater than 10 feet) or more heavily loaded walls than typical will a special analysis as shown here be necessary, even in higher-wind conditions. It is likely that the controlling factor will be a serviceability limit state (i.e., wall deflection) rather than strength, as shown in several of the floor design examples. In such cases, the wall system deflection adjustment factors of Table 5.6 should be considered.

Note:

The axial compression capacity determined above is conservative because the actual EI of the wall system is not considered in the determination of C_p for stability. No method is currently available to include system effects in the analysis of C_p; however, a K_e factor of 0.8 may be used as a reasonable assumption to determine the effective buckling length, ℓ_e, which is then used to determine C_p (see NDS•3.7.1).

Testing has demonstrated that sheathed walls like the one in this example can carry ultimate axial loads of more than 5,000 plf (NAHB/RF, 1974; other unpublished data).

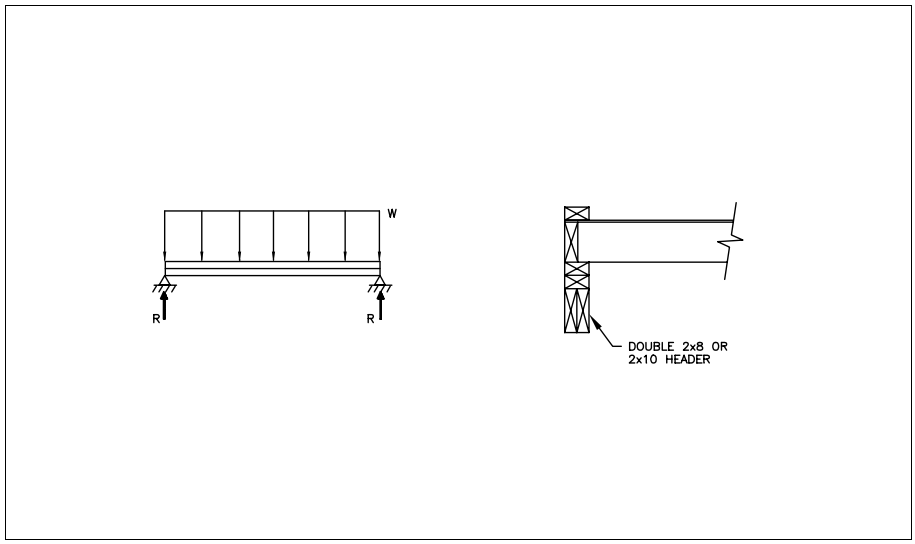
EXAMPLE 5.7 Header System Design

Given

Two-story house

Required header span = 6.3 ft (rough opening)
Species and grade = Spruce-Pine-Fir (south), No. 2
Loads on first-story header
w_{floor} = 600 plf (includes floor dead and live loads)
w_{wall} = 360 plf (includes dead, live, and snow loads supported by wall above header)*
w_{total} = 960 plf (includes dead, live, and snow loads)*
*Combined loads are determined in accordance with Table 3.1 of Chapter 3.

Find Determine header size (2x8 or 2x10) by considering system effect of all horizontal members spanning the opening.



Header System

Solution

1. Determine tabulated design values by using the NDS-S (Table 4A)

F_b = 775 psi
F_v = 70 psi
$F_{c\perp}$ = 335 psi
E = 1.1×10^6 psi

2. Determine lumber property adjustments (Section 5.2.4)

C_r = 1.3 (2x10 double header per Table 5.8)
= 1.2 (2x8 double header per Table 5.4)
C_D = 1.25 (snow load)
C_F = 1.1 (2x10)
= 1.2 (2x8)
C_H = 2.0
C_b = 1.0
C_L = 1.0 laterally supported

$F_b' = F_b C_D C_r C_F C_L = (775\ psi)(1.25)(1.3)(1.1)(1.0) = 1{,}385\ psi$ [2x10]
$= (775\ psi)(1.25)(1.2)(1.1)(1.0) = 1{,}279\ psi$ [2x8]
$F_v' = F_v C_D C_H = (70\ psi)(1.25)(2) = 175\ psi$
$F_{c\perp}' = F_{c\perp} C_b = (335 psi)(1) = 335\ psi$
$E' = E = 1.1x10^6\ psi$

With double top plate, F_b can be increased by 5 percent (Table 5.8)

$F_b' = F_b'(1.05) = 1{,}385\ psi\ (1.05) = 1{,}454\ psi$ [2x10]
$F_b' = F_b'(1.05) = 1{,}279\ psi\ (1.05) = 1{,}343\ psi$ [2x8]

3. Determine header size due to bending for floor load only

$$M_{max} = \frac{w\ell^2}{8} = \frac{(600\ plf)(6.5\ ft)^2}{8} = 3{,}169\ ft\text{-}lb$$

$$f_b = \frac{M_{max}}{S} \leq F_b'$$

$$1{,}454\ psi = \frac{3{,}169\ ft-lb\ (12\ in/ft)}{S}$$

$S = 26.2\ in^3$
S for 2 2x10 $= 2(21.39\ in^3) = 42.78\ in^3 > 26.2\ in^3$ (OK)

Try 2 2x8s

$$1{,}343\ psi = \frac{3{,}169\ ft-lb\ (12\ in/ft)}{S}$$

$S = 28.3\ in^3$
S for 2 2x8 $= 2\ (13.14) = 26.3\ in^3 < 28.3\ in^3$ (close, but no good)

4. Determine member size due to bending for combined floor and supported wall loads by using the 1.8 system factor from Table 5.8, but not explicitly calculating the load sharing with the band joist above.

$F_b' = F_b\ (C_D)(C_r)(C_F)(C_L) = 775\ psi\ (1.25)(1.8)(1.1)(1.0) = 1{,}918\ psi$

$$M_{max} = \frac{w\ell^2}{8} = \frac{(360\ plf + 600\ plf)(6.5\ ft)^2}{8} = 5{,}070\ ft\text{-}lb$$

$$f_b = M/S \leq F_b'$$

$$1{,}918\ psi = \frac{5{,}070\ ft-lb\ (12\ in/ft)}{S}$$

$S = 31.7\ in^3$
S for 2-2x10 $= 42.78\ in^3 > 31.7\ in^3$ (OK)

5. Check horizontal shear

$$V_{max} = \frac{w\ell}{2} = \frac{(600\ plf)(6.5)}{2} = 1{,}950\ lb$$

$$f_v = \frac{3V}{2A} = \frac{3(1{,}950\ lb)}{2(2)(1.5\ in)(9.25\ in)} = 106\ psi$$

$f_v \leq F_v'$

$106\ psi < 175\ psi$ (OK)

6. Check for adequate bearing

$$R_1 = R_2 = V_{max} = 1{,}950 \text{ lb}$$

$$f_{c\perp} = \frac{R}{A_b} = \frac{1{,}950 \text{ lb}}{(2)(1.5 \text{ in})(\ell_b)} = \frac{650}{\ell_b}$$

$$f_{c\perp} \leq F_{c\perp}'$$

$$\frac{650}{\ell_b} = 335$$

$$\ell_b = 1.9 \text{ in}$$ OK for bearing, use 2-2x4 jack studs (ℓ_b = 3 in)

7. Check deflection

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(600 \text{ plf})(6.5 \text{ ft})^4 (1{,}728 \text{ in}^3/\text{ft}^3)}{384(1.1\text{x}10^6 \text{ psi})[(98.9 \text{ in}^4)(2)]} = 0.11 \text{ in}$$

$$\rho_{all} = L/240 \quad \frac{(6.5 \text{ ft})(12 \text{ in/ft})}{240} = 0.325 \text{ in}$$

$$\rho_{max} < \rho_{all}$$

Conclusion

Using a system-based header design approach, a 2-2x10 header of No. 2 Spruce-Pine-Fir is found to be adequate for the 6 ft-3 in span opening. The loading condition is common to the first story of a typical two-story residential building. Using a stronger species or grade of lumber would allow the use of a 2-2x8 header. Depending on the application and potential savings, it may be more cost-effective to use the header tables found in a typical residential building code. For cost-effective ideas and concepts that allow for reduced header loads and sizes, refer to *Cost Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994). The document also contains convenient header span tables. For headers that are not part of a floor-band joist system, the design approach of this example is still relevant and similar to that used for floor girders. However, the 1.8 system factor used here would not apply, and the double top plate factor would apply only as appropriate.

EXAMPLE 5.8 *Column Design*

Given

Basement column supporting a floor girder
Spruce-Pine-Fir, No. 2 Grade
Axial design load is 4,800 lbs (D + L)
Column height is 7.3 ft (unsupported)

Find Adequacy of a 4x4 solid column

Solution

1. Determine tabulated design values by using the NDS-S (Table 4A)

F_c = 1,150 psi
E = 1.4×10^6 psi

2. Lumber property adjustments (Section 5.2.4):

C_D = 1.0
C_F = 1.15 for F_c

3. Calculate adjusted compressive capacity (NDS•3.7):

Trial 4x4

F_c^* = $F_c C_D C_F$ = 1,150 psi (1.0)(1.15) = 1,323 psi
E' = E = 1.4×10^6 psi
K_{cE} = 0.3 for visually graded
c = 0.8 for sawn lumber

$$F_{cE} = \frac{K_{cE} E'}{\left(\ell_e / d\right)^2} = \frac{0.3\,(1.4 \times 10^6 \text{psi})}{\left(7.3 \text{ ft}\,(12 \text{ in/ft}) / 3.5 \text{ in}\right)^2} = 670 \text{ psi}$$

$$C_p = \frac{1 + \left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1 + \left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

$$= \frac{1 + \left(670/1{,}323\right)}{2\,(0.8)} - \sqrt{\left[\frac{1 + \left(670/1{,}323\right)}{2\,(0.8)}\right]^2 - \frac{670/1{,}323}{0.8}} = 0.44$$

F_c' = $F_c C_D C_F C_p$ = (1,150 psi)(1.0)(1.15)(0.44) = 582 psi
P_{all} = $F'_c A$ = (582 psi)(3.5 in)(3.5 in) = 7,129 lb > 4,800 lb
OK

Conclusion

A 4x4 column is adequate for the 4,800-pound axial design load and the stated height and support conditions. In fact, a greater column spacing could be used. Note that the analysis was performed with a solid sawn column of rectangular dimension. If a nonrectangular column is used, buckling must be analyzed in the weak-axis direction in consideration of the distance between lateral supports, if any, in that direction. If a built-up column is used, it is NOT treated the same way as a solid column. Even if the dimensions are nearly the same, the built-up column is more susceptible to buckling due to slippage between adjacent members as flexure occurs in response to buckling (only if unbraced in the weak-axis direction of the built-up members). Slippage depends on how well the built-up members are fastened together, which is accounted for by the use of an additional adjustment (reduction) factor applied to the C_p equation (see Section 5.5.5 and NDS•15.3).

EXAMPLE 5.9 Simply Supported Sloped Rafter Design

Given

Two-story home
Rafter spacing 16 in on center
Rafter horizontal span is 12 ft (actual sloped span is 14.4 ft)
8:12 roof slope
Design loads (see Chapter 3):

Dead load	=	10 psf
Roof snow load	=	20 psf (20 psf ground snow)
Wind load (90 mph, gust)	=	12.7 psf (outward, uplift)
	=	7.4 psf (inward)
Roof live load	=	10 psf

Find Minimum rafter size using No. 2 Douglas-Fir-Larch (refer to Figure 5.7 for load diagram).

Solution

1. Evaluate load combinations applicable to rafter design (see Chapter 3, Table 3.1):

The load combinations to consider and initial assessment based on the magnitude of the given design loads follows

$D + (L_r \text{ or } S)$	Controls rafter design in inward-bending direction (compression side of rafter laterally supported); L_r can be ignored since the snow load magnitude is greater.
$0.6D + W_u$	May control rafter design in outward-bending direction since the compression side now has no lateral bracing unless specified; also important to rafter connections at the bearing wall and ridge beam.
$D + W$	Not controlling by inspection; gravity load D + S controls in the inward-bending direction.

2. Determine relevant lumber property values (NDS-S, Table 4A).

F_b = 900 psi
F_v = 95 psi
E = 1.6×10^6 psi

3. Determine relevant adjustments to property values assuming a 2x8 will be used (Section 5.2.4):

C_D = 1.6 (wind load combinations)
= 1.25 (snow load combination)
C_r = 1.15 (2x8, 24 inches on center)
C_H = 2.0
C_F = 1.2 (2x8)
C_L = 1.0 (inward bending, D + S, laterally braced on compression edge)
= 0.32 (outward bending, 0.6 D + W, laterally unbraced on compression edge)*

*Determined in accordance with NDS•3.3.3

$\ell_e = 1.63\,\ell_u + 3d$
$= 1.63\,(14.4\text{ ft}) + 3\,(7.25\text{ in})(1\text{ in}/12\text{ft})$
$= 25.3$ ft

$$R_B = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{(25.5\text{ ft})(12\text{ in/ft})(7.25\text{ in})}{(1.5\text{ in})^2}}$$

$= 31 < 50$ (OK)
$K_{bE} = 0.439$ (visually graded lumber)

$$F_{bE} = \frac{K_{bE} E'}{R_B^2} = \frac{0.439\,(1.6\text{x}10^6\text{ psi})}{(31)^2} = 730\text{ psi}$$

$F_b^* = F_b C_D C_r C_F$
$= 900$ psi $(1.6)(1.15)(1.2) = 1{,}987$ psi

$$C_L = \frac{1+(F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1+(F_{bE}/F_b^*)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}}$$

$C_L = 0.36$ (2x8)

4. Determine rafter transverse bending load, shear, and moment for the wind uplift load case (using Method A of Figure 5.8).

The wind load acts transverse (i.e., perpendicular) to the rafter; however, the snow load acts in the direction of gravity and must be resolved to its transverse component. Generally, the axial component of the gravity load along the rafter (which varies unknowingly depending on end connectivity) is ignored and has negligible impact considering the roof system effects that are also ignored. Also, given the limited overhang length, this too will have a negligible impact on the design of the rafter itself. Thus, the rafter can be reasonably analyzed as a sloped, simply supported bending member. In analyzing wind uplift connection forces at the outside bearing of the rafter, the designer should consider the additional uplift created by the small overhang, though for the stated condition it would amount only to about 20 pounds additional uplift load.

The net uniform uplift load perpendicular to the rafter is determined as follows:

$w_{D,\text{ transverse}} = w_D (\cos\theta)$
$= (10\text{ psf})(1.33\text{ ft})(\cos 33.7°)$
$= 11$ plf
$w_{w,\text{ transverse}} = (12.7\text{ psf})(1.33\text{ ft}) = 17$ plf (uplift)
$w_{\text{total, transverse}} = 17\text{ plf} - 11\text{ plf} = 6$ plf (net uplift)

$$\text{Shear, } V_{max} = \frac{w\ell}{2} = \frac{(6\text{ plf})(14.4\text{ ft})}{2} = 44\text{ lbs}$$

$$\text{Moment, } M_{max} = 1/8\, w\ell^2$$
$= 1/8\,(6\text{ plf})(14.4\text{ ft})^2 = 156$ ft-lb

5. Determine bending load, shear, and moment for the gravity load case (D + S) using Method B of Figure 5.8 (horizontal span):

w_D = (10 psf)(14.4 ft)(1.33 ft)/12 ft-horizontal = 16 plf
w_S = (20 psf)(12 ft)(1.33 ft)/12 ft-horizontal = 27 plf
w_{total} = 43 plf
w_{total} = (43 plf)(cos 33.7°) = 36 plf

$$\text{Shear, } V_{max} = \frac{(36 \text{ plf})(12 \text{ ft})}{2} = 216 \text{ lb}$$

$$\text{Moment, } M_{max} = 1/8\,(36 \text{ plf})(12 \text{ ft})^2 = 648 \text{ ft-lb}$$

6. Check bending stress for both loading cases and bending conditions

Outward Bending ($0.6D + W_u$)

$$f_b = \frac{M}{S} = \frac{156 \text{ ft}-\text{lb}}{13.14 \text{ in}^3}(12 \text{ in/ft}) = 142 \text{ psi}$$

$F_b' = F_b C_D C_r C_F C_L$
$= 900 \text{ psi } (1.6)(1.15)(1.2)(0.36) = 715 \text{ psi}$
$f_b << F_b'$ OK, 2x8 works and no lateral bracing of bottom compression edge is required

Inward Bending (D + S)

$$f_b = \frac{M}{S} = \frac{648 \text{ ft}-\text{lb}}{13.14 \text{ in}^3}(12 \text{ in/ft}) = 591 \text{ psi}$$

$F_b' = F_b C_D C_r C_F C_L$
$= 900 \text{ psi } (1.25)(1.15)(1.2)(1.0) = 1{,}553 \text{ psi}$
$f_b << F_b'$ (OK)

7. Check horizontal shear

V_{max} = 216 lb (see Step 5)

$$f_v = \frac{3V}{2A} = \frac{3(216 \text{ lb})}{2(1.5 \text{ in})(7.25 \text{ in})} = 30 \text{ psi}$$

$F_v' = F_v C_D C_H = 95 \text{ psi } (1.25)(2.0) = 238 \text{ psi}$
$f_v << F_v'$ (OK)

8. Check bearing

OK by inspection.

9. Check deflection criteria for gravity load condition (Section 5.2.2)

$$\rho_{all} = \frac{\ell}{180} = \frac{(14.4\ \text{ft})(12\ \text{in/ft})}{180} = 1.0\ \text{in}$$

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(36\ \text{plf})(14.4\ \text{ft})^4}{384(1.6 \times 10^6\ \text{psi})(47.6\ \text{in}^4)} \quad (1{,}728\ \text{in}^3/\text{ft}^3)$$

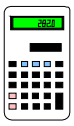
$$= 0.4\ \text{in}$$

$\rho_{max} << \rho_{all}$ (OK, usually not a mandatory roof check)

Conclusion

A 2x8, No. 2 Douglas-Fir-Larch rafter spaced at 16 inches on center was shown to have ample capacity and stiffness for the given design conditions. In fact, using a 19.2 inch on center spacing (i.e., five joists per every 8 feet) would also work with a more efficient use of lumber. It is also possible that a 2x6 could result in a reasonable rafter design for this application. For other concepts in value-added framing design, consult *Cost Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994). The document also contains prescriptive span tables for roof framing design.

EXAMPLE 5.10 Ridge Beam Design



Given

One-story building
Ridge beam span = 13 ft
Roof slope = 6:12
Rafter horizontal span = 12 ft

Loading (Chapter 3)
- Dead = 15 psf
- Snow = 20 psf
- Wind (110 mph, gust) = 6.3 psf (inward)
  = 14.2 psf (outward, uplift)
- Live = 10 psf

Find Optimum size and grade of lumber to use for a solid (single-member) ridge beam.

Solution

1. Evaluate load combinations applicable to the ridge beam design (see Chapter 3, Table 3.1)

$D + (L_r \text{ or } S)$ Controls ridge beam design in the inward-bending direction (compression side of beam laterally supported by top bearing rafters); L_r can be ignored because the roof snow load is greater.

$0.6\,D + W_u$ May control ridge beam design in outward-bending direction because the bottom (compression side) is laterally unsupported (i.e., exposed ridge beam for cathedral ceiling); also important to ridge beam connection to supporting columns. However, a ridge beam supporting rafters that are tied-down to resist wind uplift cannot experience significant uplift without significant upward movement of the rafters at the wall connection, and deformation of the entire sloped roof diaphragm (depending on roof slope).

$D + W$ Not controlling because snow load is greater in the inward direction; also, positive pressure is possible only on the sloped windward roof surface while the leeward roof surface is always under negative (suction) pressure for wind perpendicular to the ridge; the case of wind parallel to the ridge results in uplift across both sides of the roof, which is addressed in the $0.6\,D + W_u$ load combination and the roof uplift coefficients in Chapter 3 and based on this worst case wind direction.

2. Determine the ridge beam bending load, shear, and moment for the wind uplift load case

In accordance with a procedure similar to Step 4 of Example 5.9, the following ridge beam loads are determined:

Rafter sloped span = horizontal span/cos θ
= 12 ft/cos 26.6°
= 13.4 ft

Load on ridge beam

w_{dead} = (rafter sloped span)(15 psf)
[1/2 rafter span on each side]
= (13.4 ft)(15 psf)
= 201 plf

0.6 w_{dead} = 121 plf

w_{wind} = (13.4 ft)(14.2 psf) cos 26.6°
= 170 plf

w_{total} = 170 plf - 121 plf = 49 plf (outward or upward)

Shear, V_{max} = 1/2 $w\ell$ = 1/2 (49 plf)(13 ft)
= 319 lb

Moment, M_{max} = 1/8 $w\ell^2$ = 1/8 (49 plf)(13ft)^2
= 1,035 ft-lb

Note: If the rafters are adequately tied-down to resist uplift from wind, the ridge beam cannot deform upward without deforming the entire sloped roof diaphragm and the rafter-to-wall connections. Therefore, the above loads should be considered with reasonable judgment. It is more important, however, to ensure that the structure is appropriately tied together to act as a unit.

3. Determine the ridge beam loading, shear, and moment for the D + S gravity load case

D + S = 15 psf + 20 psf = 35 psf
(pressures are additive because both are gravity loads)
load on ridge beam
W_{D+S} = (13.4 ft)(35 psf) = 469 plf
Shear, V_{max} = 1/2 (469 plf)(13 ft) = 3,049 lb
Moment, M_{max} = 1/8 (469 plf)(13 ft)^2 = 9,908 ft-lb

4. Determine the optimum ridge beam size and grade based on the above bending loads and lateral support conditions.

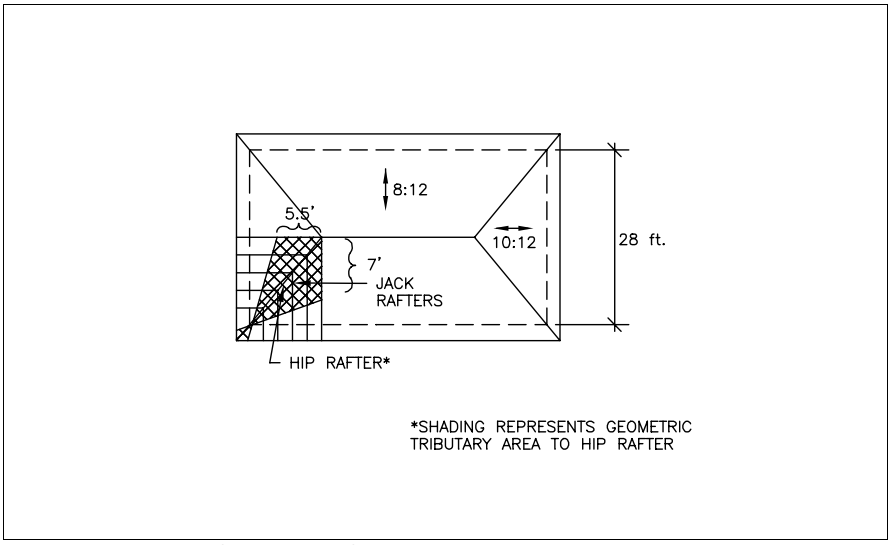
Note. The remainder of the problem is essentially identical to Example 5.9 with respect to determining the strength of the wood member. However, a trial member size and grade are needed to determine the lumber stresses as well as the lumber property adjustment values. Thus, the process of optimizing a lumber species, size, and grade selection from the multitude of choices is iterative and time consuming by hand calculation. Several computerized wood design products on the market can perform the task. However, the computerized design procedures may not allow for flexibility in design approach or assumptions if the designer is attempting to use recommendations similar to those given in this guide. For this reason, many designers prefer to create their own analysis spreadsheets as a customized personal design aid. The remainder of this problem is left to the reader for experimentation.

EXAMPLE 5.11 Hip Rafter Design

Given

One-story building
Hip rafter and roof plan as shown below
Rafters are 2x8 No. 2 Hem-Fir at 16 in on center
Loading (see Chapter 3)

Dead	=	10 psf
Snow	=	10 psf
Wind (90 mph, gust)	=	4 psf (inward)
	=	10 psf (uplift)
Live (roof)	=	15 psf



Roof Plan, Hip Rafter Framing, and Tributary Load Area

Find

1. Hip rafter design approach for rafter-ceiling joist roof framing.
2. Hip rafter design approach for cathedral ceiling framing (no cross-ties; ridge beam and hip rafter supported by end-bearing supports).

Solution

1. Evaluate load combinations applicable to the hip rafter design (see Chapter 3, Table 3.1)

By inspection, the D + L_r load combination governs the design. While the wind uplift is sufficient to create a small upward bending load above the counteracting dead load of 0.6 D, it does not exceed the gravity loading condition in effect. Since the compression edge of the hip rafter is laterally braced in both directions of strong-axis bending (i.e., jack rafters frame into the side and sheathing provides additional support to the top), the 0.6 D + W_u condition can be dismissed by inspection. Likewise, the D + W inward-bending load is considerably smaller than the gravity load condition. However, wind uplift should be considered in the design of the hip rafter connections; refer to Chapter 7.

2. Design the hip rafter for a rafter-ceiling joist roof construction (conventional practice).

Use a double 2x10 No. 2 Hem-fir hip rafter (i.e., hip rafter is one-size larger than rafters - rule of thumb). The double 2x10 may be lap-spliced and braced at or near mid-span; otherwise, a single 2x10 could be used to span continuously. The lap splice should be about 4 feet in length and both members face-nailed together with 2-10d common nails at 16 inches on center. Design is by inspection and common practice.

Note: The standard practice above applies only when the jack rafters are tied to the ceiling joists to resist outward thrust at the wall resulting from truss action of the framing system. The roof sheathing is integral to the structural capacity of the system; therefore, heavy loads on the roof before roof sheathing installation should be avoided, as is common. For lower roof slopes, a structural analysis (see next step) may be warranted because the folded-plate action of the roof sheathing is somewhat diminished at lower slopes. Also, it is important to consider connection of the hip rafter at the ridge. Usually, a standard connection using toe-nails is used, but in high wind or snow load conditions a connector or strapping should be considered.

3. Design the hip rafter by assuming a cathedral ceiling with bearing at the exterior wall corner and at a column at the ridge beam intersection

a. Assume the rafter is simply supported and ignore the negligible effect of loads on the small overhang with respect to rafter design.

b. Simplify the diamond-shaped tributary load area (see figure above) by assuming a roughly "equivalent" uniform rectangular load area as follows:

Tributary width ≈ 4 ft
w_{D+S} = (10 psf + 15 psf)(4 ft) = 100 plf

c. Determine the horizontal span of the hip rafter based on roof geometry:

$$\text{Horizontal hip span} = \sqrt{(14\,\text{ft})^2 + (11\,\text{ft})^2} = 17.8\ \text{ft}$$

d. Based on horizontal span (Method B, Figure 5.8), determine shear and bending moment:

Shear, V_{max} = 1/2 $w\ell$ = 1/2 (100 plf)(17.8 ft) = 890 lb
Moment, M_{max} = 1/8 $w\ell^2$ = 1/8 (100 plf)(17.8 ft)^2 = 3,960 ft-lb

f. Determine required section modulus assuming use of 2x12 No. 2 Hem-Fir

$$f_b = \frac{M}{S} = \frac{3{,}960\ \text{ft}-\text{lb}}{S}\ (12\ \text{in/ft}) = \frac{47{,}520\ \text{in}-\text{lb}}{S}$$

F_b' = $F_b C_D C_r C_F C_L$ (F_b from NDS-S, Table 4A)
F_b' = 850 psi (1.25)(1.0)(1.0)(1.0) = 1,063 psi
$f_b \le F_b'$

$$\frac{47{,}520\ \text{in}-\text{lb}}{S_{REQ'D}} = 1{,}063\ \text{psi}$$

$S_{REQ'D}$ = 44.7 in^3
S_{2x12} = 31.6 in^3

Therefore, 2-2x12s are required because of bending.

Try 2-2x10s,

F_b' = (850 psi)(1.25)(1.2)(1.1)(1.0) = 1,403 psi

$\frac{47{,}520 \text{ in} - \text{lb}}{S_{REQ'D}}$ = 1,403 psi

$S_{REQ'D}$ = 34 in^3

S_{2x10} = 21.39 in^3

Therefore, 2-2x10s are acceptable (2x21.39 in^3 = 42.8 in^3).

g. Check horizontal shear:

$$f_v = \frac{3V}{2A} = \frac{3\ (890\ \text{lb})}{2\ (2)(1.5\ \text{in})(9.25\ \text{in})} = 48.1 \text{ psi}$$

$f_v << F_v'$

OK by inspection

h. Consider deflection:

Deflection is OK by inspection. No method exists to accurately estimate deflection of a hip rafter that is subject to significant system stiffness because of the folded-plate action of the roof sheathing diaphragm.

Conclusion

Use 2-2x10 (No. 2 Hem-Fir) for the hip rafters for the cathedral ceiling condition (not considering sloped roof sheathing system effects). However, a cathedral ceiling with a hip roof is not a common occurrence. For traditional rafter-ceiling joist roof construction, a hip rafter one or two sizes larger than the rafters can be used, particularly if it is braced at or near mid-span. With a ceiling joist or cross-ties, the ridge member and hip rafter member need only serve as plates or boards that provide a connection interface, not a beam, for the rafters.

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CHAPTER 6

Lateral Resistance to Wind and Earthquake

6.1 General

The objectives in designing a building's lateral resistance to wind and earthquake forces are

- to provide a system of shear walls, diaphragms, and interconnections to transfer lateral loads and overturning forces to the foundation;
- to prevent building collapse in extreme wind and seismic events; and
- to provide adequate stiffness to the structure for service loads experienced in moderate wind and seismic events.

In light-frame construction, the lateral force-resisting system (LFRS) comprises shear walls, diaphragms, and their interconnections to form a whole-building system that may behave differently than the sum of its individual parts. In fact, shear walls and diaphragms are themselves subassemblies of many parts and connections. Thus, designing an efficient LFRS system is perhaps the greatest challenge in the structural design of light-frame buildings. In part, the challenge results from the lack of any single design methodology or theory that provides reasonable predictions of complex, large-scale system behavior in conventionally built or engineered light-frame buildings.

Designer judgment is a crucial factor that comes into play when the designer selects how the building is to be analyzed and to what extent the analysis should be assumed to be a correct representation of the true design problem. Designer judgment is essential in the early stages of design because the analytic methods and assumptions used to evaluate the lateral resistance of light-frame buildings are not in themselves correct representations of the problem. They are

analogies that are sometimes reasonable but at other times depart significantly from reason and actual system testing or field experience.

This chapter focuses on methods for evaluating the lateral resistance of individual subassemblies of the LFRS (i.e., shear walls and diaphragms) and the response of the whole building to lateral loads (i.e., load distribution). Traditional design approaches as well as innovative methods, such as the *perforated shear wall design method*, are integrated into the designer's "tool box." While the code-approved methods have generally "worked," there is considerable opportunity for improvement and optimization. Therefore, the information and design examples presented in this chapter provide a useful guide and resource that supplement existing building code provisions. More important, the chapter is aimed at fostering a better understanding of the role of analysis versus judgment and promoting more efficient design in the form of alternative methods.

The lateral design of light-frame buildings is not a simple endeavor that provides "exact" solutions. By the very nature of the LFRS, the real behavior of light-frame buildings is highly dependent on the performance of building systems, including the interactions of structural and nonstructural components. For example, the nonstructural components in conventional housing (i.e., sidings, interior finishes, interior partition walls, and even windows and trim) can account for more than 50 percent of a building's lateral resistance. Yet, the contribution of these components is not considered as part of the "designed" LFRS for lack of appropriate design tools and building code provisions that may prohibit such considerations. In addition, the need for simplified design methods inevitably leads to a trade-off–analytical simplicity for design efficiency.

In seismic design, factors that translate into better performance may not always be obvious. The designer should become accustomed to thinking in terms of the relative stiffness of components that make up the whole building. Important, too, is an understanding of the inelastic (nonlinear), nonrigid body behavior of wood-framed systems that affect the optimization of strength, stiffness, dampening, and ductility. In this context, the concept that more strength is better is insupportable without considering the impact on other important factors. Many factors relate to a structural system's deformation capability and ability to absorb and safely dissipate energy from abusive cyclic motion in a seismic event. The intricate interrelationship of these several factors is difficult to predict with available seismic design approaches.

For example, the basis for the seismic response modifier R is a subjective representation of the behavior of a given structure or structural system in a seismic event (refer to Chapter 3). In a sense, it bears evidence of the inclusion of "fudge factors" in engineering science for reason of necessity (not of preference) in attempting to mimic reality. It is not necessarily surprising, then, that the amount of wall bracing in conventional homes shows no apparent correlation with the damage levels experienced in seismic events (HUD, 1999). Similarly, the near-field damage to conventional homes in the Northridge Earthquake did not correlate with the magnitude of response spectral ground accelerations in the short period range (HUD, 1999). The short-period spectral response acceleration, it will be recalled, is the primary ground motion parameter used in the design of most low-rise and light-frame buildings (refer to Chapter 3).

The apparent lack of correlation between design theory and actual outcome points to the tremendous uncertainty in existing seismic design methods

for light-frame structures. In essence, a designer's compliance with accepted seismic design provisions may not necessarily be a good indication of actual performance in a major seismic event. This statement may be somewhat unsettling but is worthy of mention. For wind design, the problem is not as severe in that the lateral load can be more easily treated as a static load, with system response primarily a matter of determining lateral capacity without complicating inertial effects, at least for small light-frame buildings.

In conclusion, the designer should have a reasonable knowledge of the underpinnings of current LFRS design approaches (including their uncertainties and limitations). However, many designers do not have the opportunity to become familiar with the experience gained from testing whole buildings or assemblies. Design provisions are generally based on an "element-based" approach to engineering and usually provide little guidance on the performance of the various elements as assembled in a real building. Therefore, the next section presents a brief overview of several whole-house lateral load tests.

6.2 Overview of Whole-Building Tests

A growing number of full-scale tests of houses have been conducted to gain insight into actual system strength and structural behavior. Several researchers have recently summarized the body of research; the highlights follow (Thurston, 1994; NIST, 1998).

One whole-house test program investigated the lateral stiffness and natural frequency of a production-built home (Yokel, Hsi, and Somes, 1973). The study applied a design load simulating a uniform wind pressure of 25 psf to a conventionally built home: a two-story, split-foyer dwelling with a fairly typical floor plan. The maximum deflection of the building was only 0.04 inches and the residual deflection about 0.003 inches. The natural frequency and dampening of the building were 9 hz (0.11 s natural period) and 6 percent, respectively. The testing was nondestructive such that the investigation yielded no information on "postyielding" behavior; however, the performance was good for the nominal lateral design loads under consideration.

Another whole-house test applied transverse loads without uplift to a wood-framed house. Failure did not occur until the lateral load reached the "equivalent" of a 220 mph wind event without inclusion of uplift loads (Tuomi and McCutcheon, 1974). The house was fully sheathed with 3/8-inch plywood panels, and the number of openings was somewhat fewer than would be expected for a typical home (at least on the street-facing side). The failure took the form of slippage at the floor connection to the foundation sill plate (i.e., there was only one 16d toenail at the end of each joist, and the band joist was not connected to the sill). The connection was somewhat less than what is now required in the United States for conventional residential construction (ICC, 1998). The racking stiffness of the walls nearly doubled from that experienced before the addition of the roof framing. In addition, the simple 2x4 wood trusses were able to carry a gravity load of 135 psf–more than three times the design load of 40 psf. However, it is important to note that combined uplift and lateral load, as would be expected in high-wind conditions, was not tested. Further, the test house was relatively small and "boxy" in comparison to modern homes.

Many whole-house tests have been conducted in Australia. In one series of whole-house tests, destructive testing has shown that conventional residential construction (only slightly different from that in the United States) was able to withstand 2.4 times its intended design wind load (corresponding to a 115 mph wind speed) without failure of the structure (Reardon and Henderson, 1996). The test house had typical openings for a garage, doors, and windows, and no special wind-resistant detailing. The tests applied a simultaneous roof uplift load of 1.2 times the total lateral load. The drift in the two-story section was 3 mm at the maximum applied load while the drift in the open one-story section (i.e., no interior walls) was 3 mm at the design load and 20 mm at the maximum applied load.

Again in Australia, a house with fiber cement exterior cladding and plasterboard interior finishes was tested to 4.75 times its "design" lateral load capacity (Boughton and Reardon, 1984). The walls were restrained with tie rods to resist wind uplift loads as required in Australia's typhoon-prone regions. The roof and ceiling diaphragm was found to be stiff; in fact, the diaphragm rigidly distributed the lateral loads to the walls. The tests suggested that the house had sufficient capacity to resist a design wind speed of 65 m/s (145 mph).

Yet another Australian test of a whole house found that the addition of interior ceiling finishes reduced the deflection (i.e., drift) of one wall line by 75 percent (Reardon, 1988; Reardon, 1989). When cornice trim was added to cover or dress the wall-ceiling joint, the deflection of the same wall was reduced by another 60 percent (roughly 16 percent of the original deflection). The tests were conducted at relatively low load levels to determine the impact of various nonstructural components on load distribution and stiffness.

Recently, several whole-building and assembly tests in the United States have been conducted to develop and validate sophisticated finite-element computer models (Kasal, Leichti, and Itani, 1994). Despite some advances in developing computer models as research tools, the formulation of a simplified methodology for application by designers lags behind. Moreover, the computer models tend to be time-intensive to operate and require detailed input for material and connection parameters that would not normally be available to typical designers. Given the complexity of system behavior, the models are often not generally applicable and require "recalibration" whenever new systems or materials are specified.

In England, researchers have taken a somewhat different approach by moving directly from empirical system data to a simplified design methodology, at least for shear walls (Griffiths and Wickens, 1996). This approach applies various "system factors" to basic shear wall design values to obtain a value for a specific application. System factors account for material effects in various wall assemblies, wall configuration effects (i.e., number of openings in the wall), and interaction effects with the whole building. One factor even accounts for the fact that shear loads on wood-framed shear walls in a full brick-veneered building are reduced by as much as 45 percent for wind loads, assuming, of course, that the brick veneer is properly installed and detailed to resist wind pressures.

More recently, whole-building tests have been conducted in Japan (and to a lesser degree in the United States) by using large-scale shake tables to study the inertial response of whole, light-frame buildings (Yasumura, 1999). The tests have demonstrated whole-building stiffness of about twice that experienced by

walls tested independently. The results are reasonably consistent with those reported above. Apparently, many whole-building tests have been conducted in Japan, but the associated reports are available only in Japanese (Thurston, 1994).

The growing body of whole-building test data will likely improve the understanding of the actual performance of light-frame structures in seismic events to the extent that the test programs are able to replicate actual conditions. Actual performance must also be inferred from anecdotal experience or, preferably, from experimentally designed studies of buildings experiencing major seismic or wind events (refer to Chapter 1).

6.3 LFRS Design Steps and Terminology

The lateral force resisting system (LFRS) of a home is the "whole house" including practically all structural and non-structural components. To enable a rational and tenable design analysis, however, the complex structural system of a light-frame house is usually subjected to many simplifying assumptions; refer to Chapter 2. The steps required for thoroughly designing a building's LFRS are outlined below in typical order of consideration:

1. Determine a building's architectural design, including layout of walls and floors (usually pre-determined).
2. Calculate the lateral loads on the structure resulting from wind and/or seismic conditions (refer to Chapter 3).
3. Distribute shear loads to the LFRS (wall, floor, and roof systems) based on one of the design approaches described later in this chapter (refer to Section 6.4.1).
4. Determine *shear wall* and *diaphragm* assembly requirements for the various LFRS components (sheathing thickness, fastening schedule, etc.) to resist the stresses resulting from the applied lateral forces (refer to Section 6.5).
5. Design the *hold-down restraints* required to resist overturning forces generated by lateral loads applied to the vertical components of the LFRS (i.e., shear walls).
6. Determine interconnection requirements to transfer shear between the LFRS components (i.e., roof, walls, floors, and foundation).
7. Evaluate *chords* and *collectors* (or *drag struts*) for adequate capacity and for situations requiring special detailing such as splices.

It should be noted that, depending on the method of distributing shear loads (refer to Section 6.4.1), Step 3 may be considered a preliminary design step. If, in fact, loads are distributed according to stiffness in Step 3, then the LFRS must already be defined; therefore, the above sequence can become iterative between Steps 3 and 4. A designer need not feel compelled to go to such a level of complexity (i.e., using a stiffness-based force distribution) in designing a simple home, but the decision becomes less intuitive with increasing plan complexity.

The above list of design steps introduced several terms that are defined below.

Horizontal diaphragms are assemblies such as the roof and floors that act as "deep beams" by collecting and transferring lateral forces to the *shear walls*, which are the vertical components of the LFRS. The diaphragm is analogous to a horizontal, simply supported beam laid flatwise; a shear wall is analogous to a vertical, fixed-end, cantilevered beam. Chapter 2 discussed the function of the LFRS and the lateral load path. The reader is referred to that chapter for a conceptual overview of the LFRS and to Chapter 3 for methodologies to calculate lateral loads resulting from wind and earthquake forces.

Chords are the members (or a system of members) that form a "flange" to resist the tension and compression forces generated by the "beam" action of a diaphragm or shear wall. As shown in Figure 6.1, the chord members in shear walls and diaphragms are different members, but they serve the same purpose in the beam analogy. A *collector* or *drag strut*, which is usually a system of members in light-frame buildings, "collects" and transfers loads by tension or compression to the shear resisting segments of a wall line (see Figure 6.2a).

In typical light-frame homes, special design of chord members for floor diaphragms may involve some modest detailing of splices at the diaphragm boundary (i.e., joints in the band joists). If adequate connection is made between the band joist and the wall top plate, then the diaphragm sheathing, band joists, and wall framing function as a "composite" chord in resisting the chord forces. Thus, the diaphragm chord is usually integral with the collectors or drag struts in shear walls. Given that the collectors on shear walls often perform a dual role as a chord on a floor or roof diaphragm boundary, the designer needs only to verify that the two systems are reasonably interconnected along their boundary, thus ensuring composite action as well as direct shear transfer (i.e., slip resistance) from the diaphragm to the wall. As shown in Figure 6.2b, the failure plane of a typical "composite" collector or diaphragm chord can involve many members and their interconnections.

For shear walls in typical light-frame buildings, tension and compression forces on shear wall chords are usually considered. In particular, the connection of hold-downs to shear wall chords should be carefully evaluated with respect to the transfer of tension forces to the structure below. Tension forces result from the overturning action (i.e., overturning moment) caused by the lateral shear load on the shear wall. In some cases, the chord may be required to be a thicker member to allow for an adequate hold-down connection or to withstand the tension and compression forces presumed by the beam analogy. Fortunately, most chords in light-frame shear walls are located at the ends of walls or adjacent to openings where multiple studs are already required for reasons of constructability and gravity load resistance (see cross-section "B" in Figure 6.1).

FIGURE 6.1 *Chords in Shear Walls and Horizontal Diaphragms Using the "Deep Beam" Analogy*

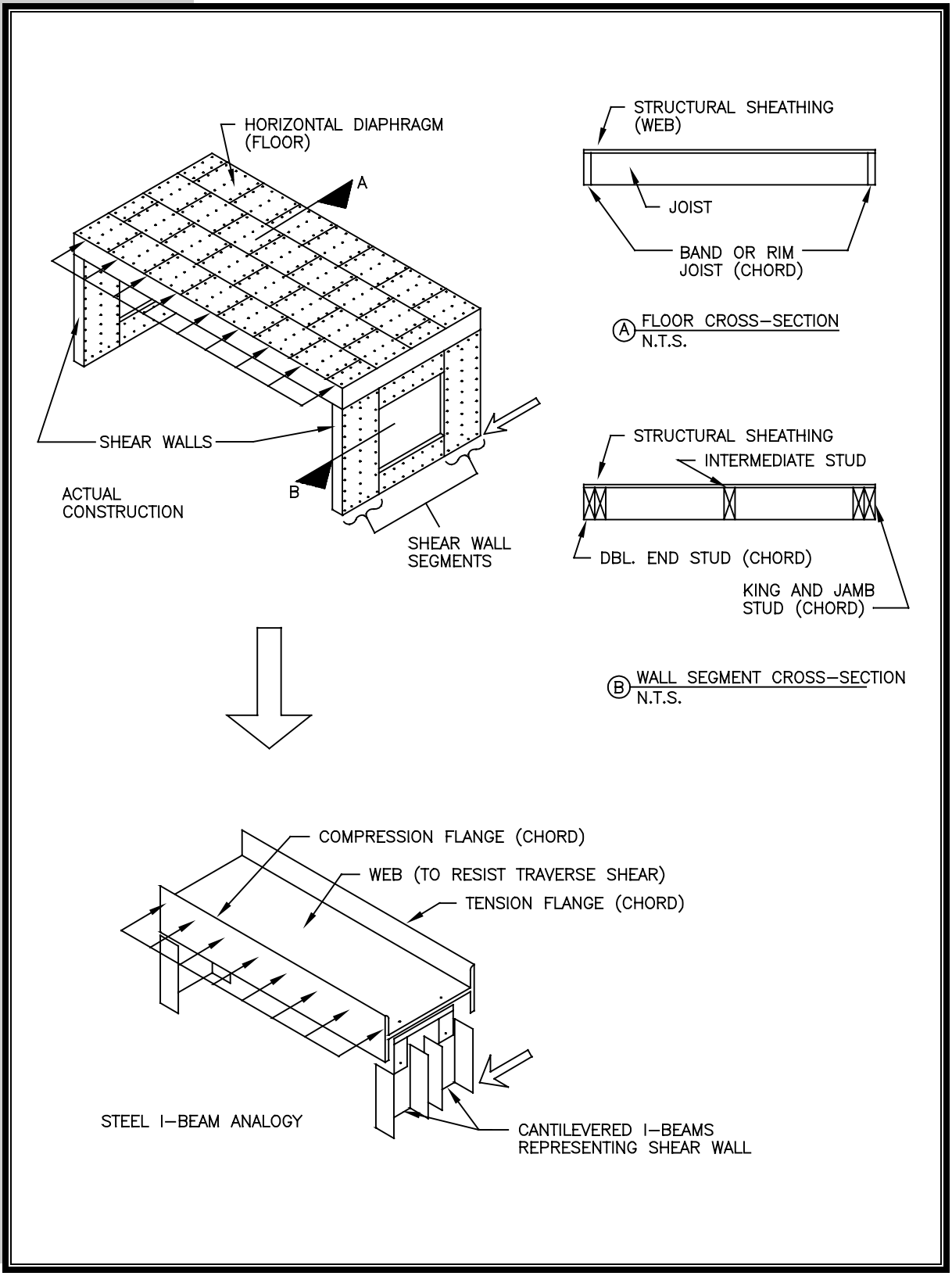
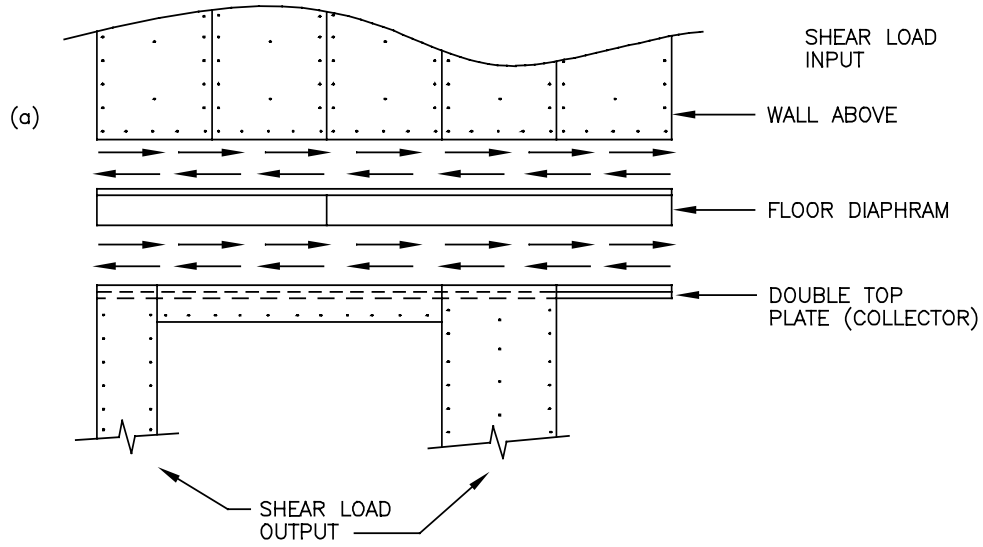
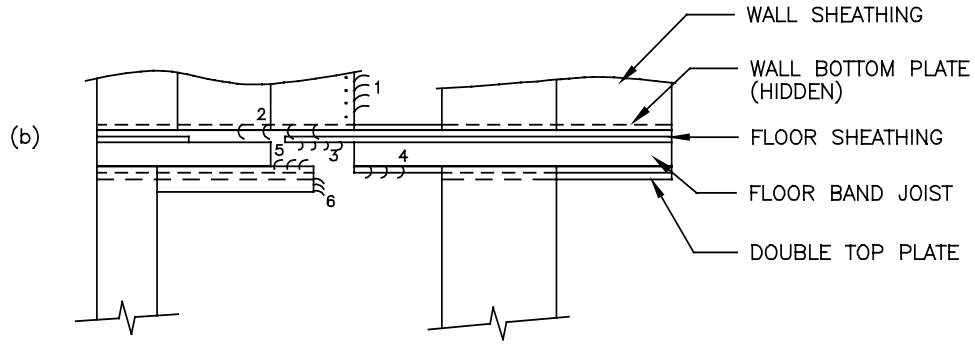


FIGURE 6.2 *Shear Wall Collector and the Composite Failure Plane (Failure plane also applies to diaphragm chords)*





COMPOSITE FAILURE PLANE FOR A COLLECTOR:

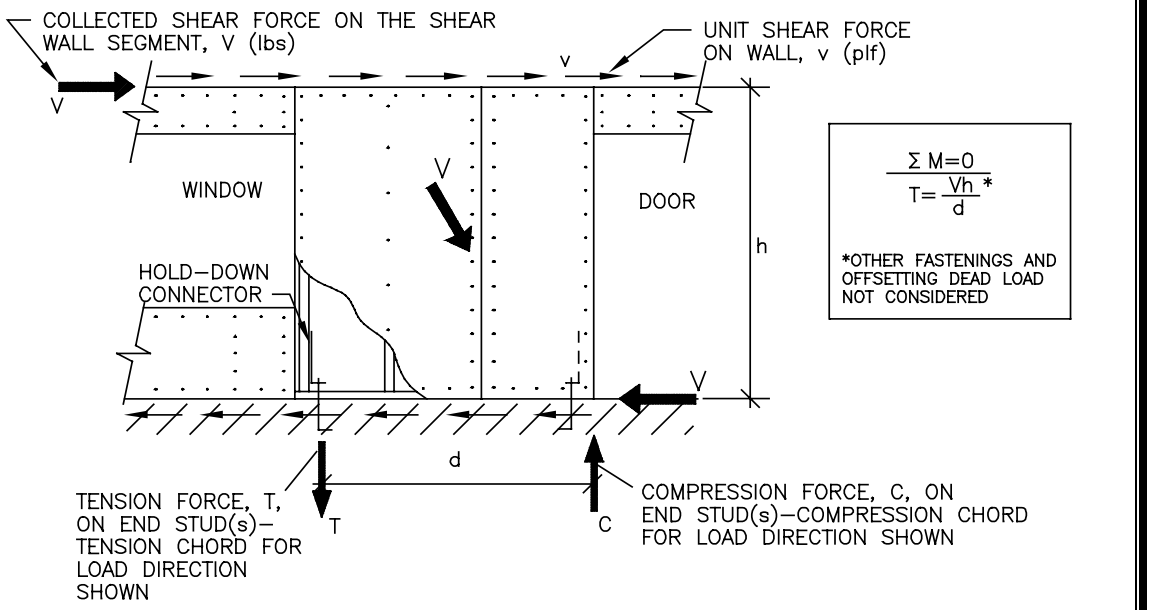
1. SHEATHING FASTENERS AT COMMON WALL STUD
2. WALL BOTTOM PLATE FASTENERS
3. FLOOR SHEATHING FASTENERS
4. DOUBLE TOP PLATE SPLICE NAILS
5. SHEATHING FASTENERS AT REGION OF DOUBLE TOP PLATE SPLICE
6. SHEATHING AND HEADER FASTENERS TO KING STUD

Hold-down restraints are devices used to restrain the whole building and individual shear wall segments from the overturning that results from the leveraging (i.e., overturning moment) created by lateral forces. The current engineering approach calls for restraints that are typically metal connectors (i.e., straps or brackets) that attach to and anchor the chords (i.e., end studs) of shear wall segments (see Figure 6.3a). In many typical residential applications, however, overturning forces may be resisted by the dead load and the contribution of many component connections (see Figure 6.3b). Unfortunately (but in reality), this consideration may require a more intensive analytic effort and greater degree of designer presumption because overturning forces may disperse through many "load paths" in a nonlinear fashion. Consequently, the analysis of overturning becomes much more complicated; the designer cannot simply assume a single load path through a single hold-down connector. Indeed, analytic knowledge of overturning has not matured sufficiently to offer an exact performance-based solution, even though experience suggests that the resistance provided by conventional framing has proven adequate to prevent collapse in all but the most extreme conditions or mis-applications (see Chapter 1 and Section 6.2).

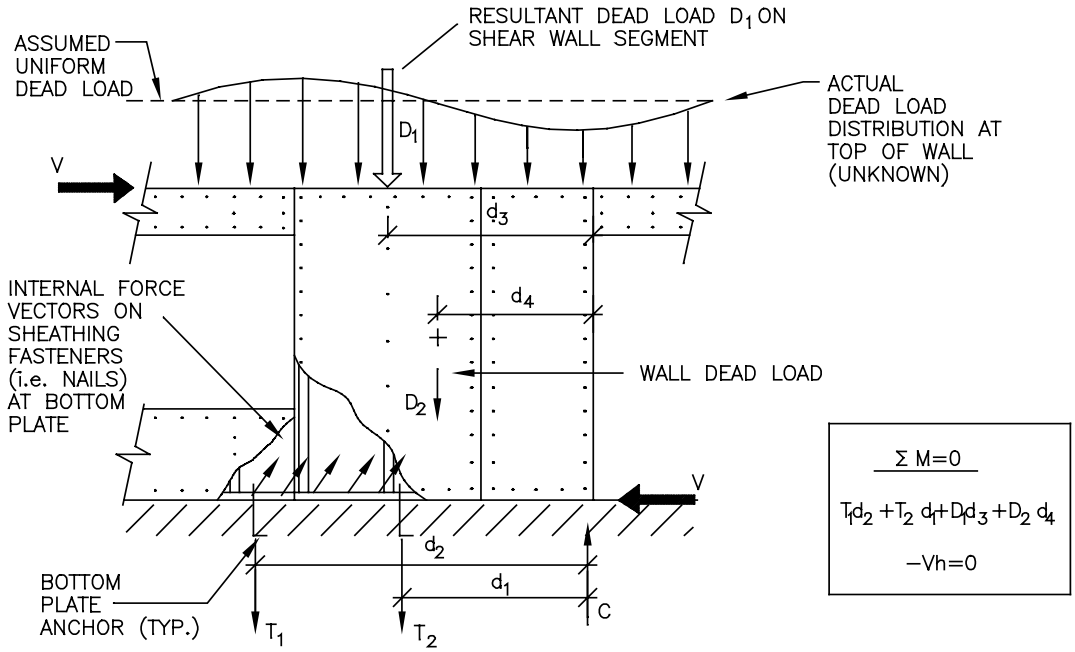
Framing and fastenings at wall corner regions are a major factor in explaining the actual behavior of conventionally built homes, yet there is no currently recognized way to account for this effect from a performance-based design perspective. Several studies have investigated corner framing effects in restraining shear walls without the use of hold-down brackets. In one such study, cyclic and monotonic tests of typical 12-foot-long wood-framed shear walls with 2- and 4-foot corner returns have demonstrated that overturning forces can be resisted by reasonably detailed corners (i.e., sheathing fastened to a common corner stud), with the reduction in shear capacity only about 10 percent from that realized in tests of walls with hold-downs instead of corner returns (Dolan and Heine, 1997c). The corner framing approach can also improve ductility (Dolan and Heine, 1997c) and is confirmed by testing in other countries (Thurston, 1994). In fact, shear wall test methods in New Zealand use a simple three-nail connection to provide hold-down restraint (roughly equivalent to three 16d common nails in a single shear wood-to-wood connection with approximately a 1,200- to 1,500-pound ultimate capacity). The three-nail connection resulted from an evaluation of the restraining effect of corners and the selection of a minimum value from typical construction. The findings of the tests reported above do not consider the beneficial contribution of the dead load in helping to restrain a corner from uplift as a result of overturning action.

The discussion to this point has given some focus to conventional residential construction practices for wall bracing that have worked effectively in typical design conditions. This observation is a point of contention, however, because conventional construction lacks the succinct loads paths that may be assumed when following an accepted engineering method. Therefore, conventional residential construction does not lend itself readily to current engineering conventions of analyzing a lateral force resisting system in light-frame construction. As a result, it is difficult to define appropriate limitations to the use of conventional construction practices based purely on existing conventions of engineering analysis.

FIGURE 6.3 *Two Types of Hold-Down Restraint and Basic Analytic Concepts*



(a) DISCRETE HOLD–DOWN CONNECTOR FOR OVERTURNING RESTRAINT (SIMPLIFIED ANALYSIS)



(b) SYSTEM OF FASTENINGS AND DEAD LOAD FOR OVERTURNING RESTRAINT AND SHEAR TRANSFER (COMPLEX ANALYSIS)

6.4 The Current LFRS Design Practice

This section provides a brief overview of the current design practices for analyzing the LFRS of light-frame buildings. It highlights the advantages and disadvantages of the various approaches but, in the absence of a coherent body of evidence, makes no attempt to identify which approach, if any, may be considered superior. Where experience from whole-building tests and actual building performance in real events permits, the discussion provides a critique of current design practices that, for lack of better methods, relies somewhat on an intuitive sense for the difference between the structure as it is analyzed and the structure as it may actually perform. The intent is not to downplay the importance of engineering analysis; rather, the designer should understand the implications of the current analytic methods and their inherent assumptions and then put them into practice in a suitable manner.

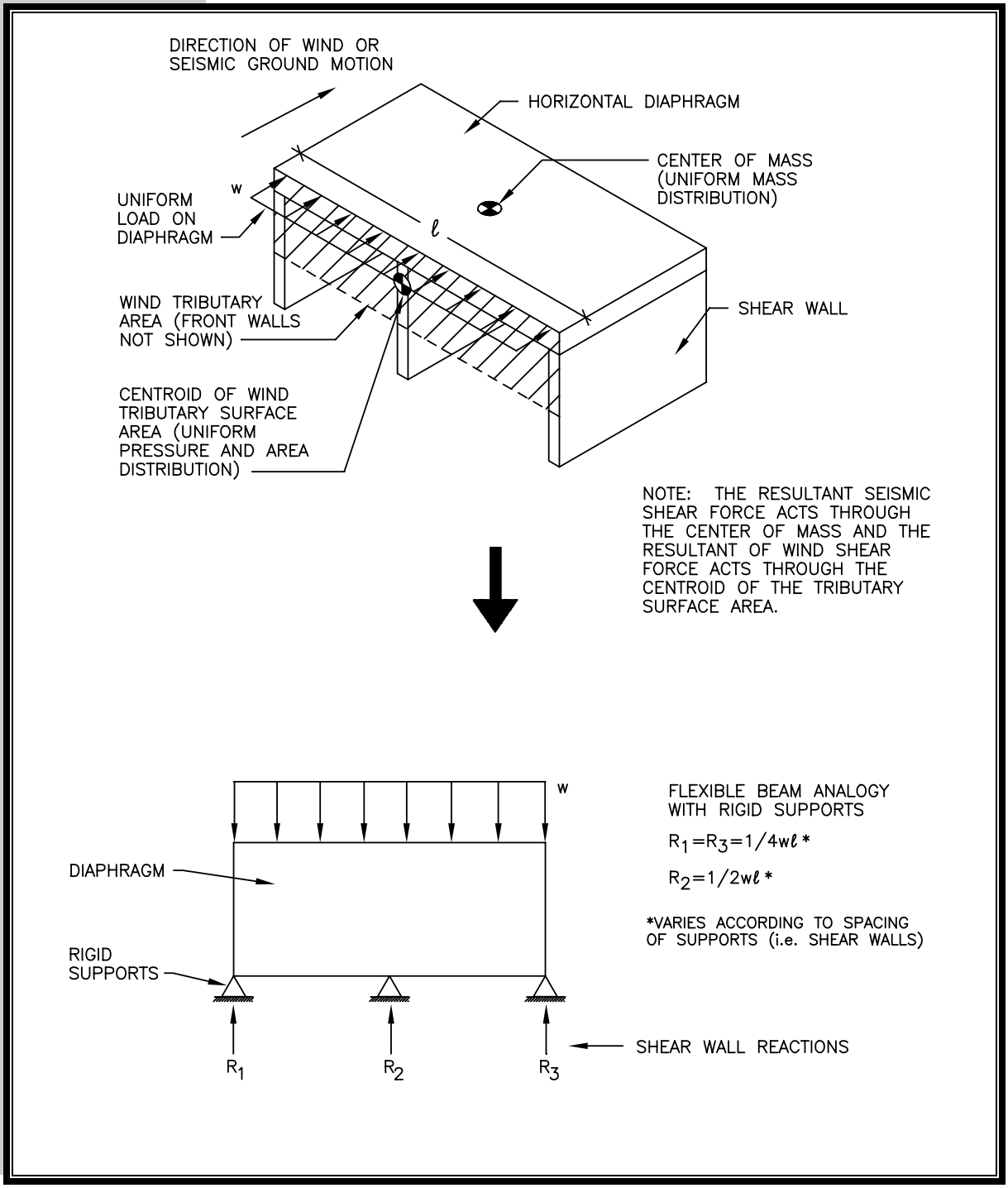
6.4.1 Lateral Force Distribution Methods

The design of the LFRS of light-frame buildings generally follows one of three methods described below. Each differs in its approach to distributing whole-building lateral forces through the horizontal diaphragms to the shear walls. Each varies in the level of calculation, precision, and dependence on designer judgment. While different solutions can be obtained for the same design by using the different methods, one approach is not necessarily preferred to another. All may be used for the distribution of seismic and wind loads to the shear walls in a building. However, some of the most recent building codes may place limitations or preferences on certain methods.

Tributary Area Approach (Flexible Diaphragm)

The *tributary area approach* is perhaps the most popular method used to distribute lateral building loads. Tributary areas based on building geometry are assigned to various components of the LFRS to determine the wind or seismic loads on building components (i.e., shear walls and diaphragms). The method assumes that a diaphragm is relatively flexible in comparison to the shear walls (i.e., a “flexible diaphragm”) such that it distributes forces according to tributary areas rather than according to the stiffness of the supporting shear walls. This hypothetical condition is analogous to conventional beam theory, which assumes rigid supports as illustrated in Figure 6.4 for a continuous horizontal diaphragm (i.e., floor) with three supports (i.e., shear walls).

FIGURE 6.4 *Lateral Force Distribution by a "Flexible" Diaphragm (tributary area approach)*



In seismic design, tributary areas are associated with uniform area weights (i.e., dead loads) assigned to the building systems (i.e., roof, walls, and floors) that generate the inertial seismic load when the building is subject to lateral ground motion (refer to Chapter 3 on earthquake loads). In wind design, the tributary areas are associated with the lateral component of the wind load acting on the exterior surfaces of the building (refer to Chapter 3 on wind loads).

The flexibility of a diaphragm depends on its construction as well as on its aspect ratio (length:width). Long, narrow diaphragms, for example, are more flexible in bending along the their long dimension than short, wide diaphragms. In other words, rectangular diaphragms are relatively stiff in one loading direction and relatively flexible in the other. Similarly, long shear walls with few openings are stiffer than walls comprised of only narrow shear wall segments. While analytic methods are available to calculate the stiffness of shear wall segments and diaphragms (refer to Section 6.5), the actual stiffness of these systems is extremely difficult to predict accurately (refer to Section 6.2). It should be noted that if the diaphragm is considered infinitely rigid relative to the shear walls and the shear walls have roughly equivalent stiffness, the three shear wall reactions will be roughly equivalent (i.e., $R_1 = R_2 = R_3 = 1/3[w][l]$). If this assumption were more accurate, the interior shear wall would be overdesigned and the exterior shear walls underdesigned with use of the tributary area method. In many cases, the correct answer is probably somewhere between the apparent over- and under-design conditions.

The tributary area approach is reasonable when the layout of the shear walls is generally symmetrical with respect to even spacing and similar strength and stiffness characteristics. It is particularly appropriate in concept for simple buildings with diaphragms supported by two exterior shear wall lines (with similar strength and stiffness characteristics) along both major building axes. More generally, the major advantages of the tributary area LFRS design method are its simplicity and applicability to simple building configurations. In more complex applications, the designer should consider possible imbalances in shear wall stiffness and strength that may cause or rely on torsional response to maintain stability under lateral load (see *relative stiffness design approach*).

Total Shear Approach ("Eyeball" Method)

Considered the second most popular and simplest of the three LFRS design methods, the *total shear approach* uses the total story shear to determine a total amount of shear wall length required on a given story level for each orthogonal direction of loading. The amount of shear wall is then "evenly" distributed in the story according to designer judgment. While the total shear approach requires the least amount of computational effort among the three methods, it demands good "eyeball" judgment as to the distribution of the shear wall elements in order to address or avoid potential loading or stiffness imbalances. In seismic design, loading imbalances may be created when a building's mass distribution is not uniform. In wind design, loading imbalances result when the surface area of the building is not uniform (i.e., taller walls or steeper roof sections experience greater lateral wind load). In both cases, imbalances are created when the center of resistance is offset from either the center of mass (seismic design) or the resultant force center of the exterior surface pressures (wind design). Thus, the reliability of the total shear approach is highly dependent on the designer's judgment and intuition regarding load distribution and structural response. If used indiscriminately without consideration of the above factors, the total shear approach to LFRS design can result in poor performance in severe seismic or wind events. However, for small structures such

as homes, the method has produced reasonable designs, especially in view of the overall uncertainty in seismic and wind load analysis.

Relative Stiffness Design Approach (Rigid Diaphragm)

The *relative stiffness approach* was first contemplated for house design in the 1940s and was accompanied by an extensive testing program to create a database of racking stiffnesses for a multitude of interior and exterior wall constructions used in residential construction at that time (NBS, 1948). If the horizontal diaphragm is considered stiff relative to the shear walls, then the lateral forces on the building are distributed to the shear wall lines according to their relative stiffness. A stiff diaphragm may then rotate some degree to distribute loads to all walls in the building, not just to walls parallel to an assumed loading direction. Thus, the relative stiffness approach considers torsional load distribution as well as distribution of the direct shear loads. When torsional force distribution needs to be considered, whether to demonstrate lateral stability of an "unevenly" braced building or to satisfy a building code requirement, the relative stiffness design approach is the only available option.

Although the approach is conceptually correct and comparatively more rigorous than the other two methods, its limitations with respect to reasonably determining the real stiffness of shear wall lines (composed of several restrained and unrestrained segments and nonstructural components) and diaphragms (also affected by nonstructural components and the building plan configuration) render its analogy to actual structural behavior uncertain. Ultimately, it is only as good as the assumptions regarding the stiffness or shear walls and diaphragms relative to the actual stiffness of a complete building system. As evidenced in the previously mentioned whole-building tests and in other authoritative design texts on the subject (Ambrose and Vergun, 1987), difficulties in accurately predicting the stiffness of shear walls and diaphragms in actual buildings are significant. Moreover, unlike the other methods, the relative stiffness design approach is iterative in that the distribution of loads to the shear walls requires a preliminary design so that relative stiffness may be estimated. One or more adjustments and recalculations may be needed before reaching a satisfactory final design.

However, it is instructional to consider analytically the effects of stiffness in the distribution of lateral forces in an LFRS, even if based on somewhat idealized assumptions regarding relative stiffness (i.e., diaphragm is rigid over the entire expanse of shear walls). The approach is a reasonable tool when the torsional load distribution should be considered in evaluating or demonstrating the stability of a building, particularly a building that is likely to undergo significant torsional response in a seismic event. Indeed, torsional imbalances exist in just about any building and may be responsible for the relatively good performance of some light-frame homes when one side (i.e., the street-facing side of the building) is weaker (i.e., less stiff and less strong) than the other three sides of the building. This condition is common owing to the aesthetic desire and functional need for more openings on the front side of a building. However, a torsional response in the case of underdesign (i.e., "weak" or "soft" story) can wreak havoc on a building and constitute a serious threat to life.

6.4.2 Shear Wall Design Approaches

Once the whole-building lateral loads have been distributed and assigned to the floor and roof diaphragms and various designated shear walls, each of these subassemblies must be designed to resist the assigned shear loads. As discussed, the whole-building shear loads are distributed to various shear walls ultimately in accordance with the principle of relative stiffness (whether handled by judgment, analytic assumptions per a selected design method, or both). Similarly, the distribution of the assigned shear load to the various shear wall segments within a given shear wall line is based on the same principle, but at a different scale. The scale is the subassembly (or shear wall) as opposed to the whole building.

The methods for designing and distributing the forces within a shear wall line differ as described below. As with the three different approaches described for the distribution of lateral building loads, the shear wall design methods place different levels of emphasis on analytic rigor and judgment. Ultimately, the configuration of the building (i.e., are the walls inherently broken into individual segments by large openings or many offsets in plan dimensions?) and the required demand (i.e., shear load) should drive the choice of a shear wall design approach and the resulting construction detailing. Thus, the choice of which design method to use is a matter of designer judgment and required performance. In turn, the design method itself imposes detailing requirements on the final construction in compliance with the analysis assumptions. Accordingly, the above decisions affect the efficiency of the design effort and the complexity of the resulting construction details.

Segmented Shear Wall (SSW) Design Approach

The *segmented shear wall design approach*, well recognized as a standard design practice, is the most widely used method of shear wall design. It considers the shear resisting segments of a given shear wall line as separate "elements," with each segment restrained against overturning by the use of hold-down connectors at its ends. Each segment is a fully sheathed portion of the wall without any openings for windows or doors. The design shear capacity of each segment is determined by multiplying the length of the segment (sometimes called segment width) by tabulated unit shear design values that are available in the building codes and newer design standards. In its simplest form, the approach analyzes each shear wall segment for static equilibrium in a manner analogous to a cantilevered beam with a fixed end (refer to Figures 6.1 and 6.3a). In a wall with multiple designated shear wall segments, the typical approach to determining an adequate total length of all shear wall segments is to divide the design shear load demand on the wall by the unit shear design value of the wall construction. The effect of stiffness on the actual shear force distribution to the various segments is simply handled by complying with code-required maximum shear wall segment aspect ratios (i.e., segment height divided by segment width). Although an inexact and circuitous method of handling the problem of shear force distribution in a shear wall line, the SSW approach has been in successful practice for many years, partly due to the use of conservative unit shear design values.

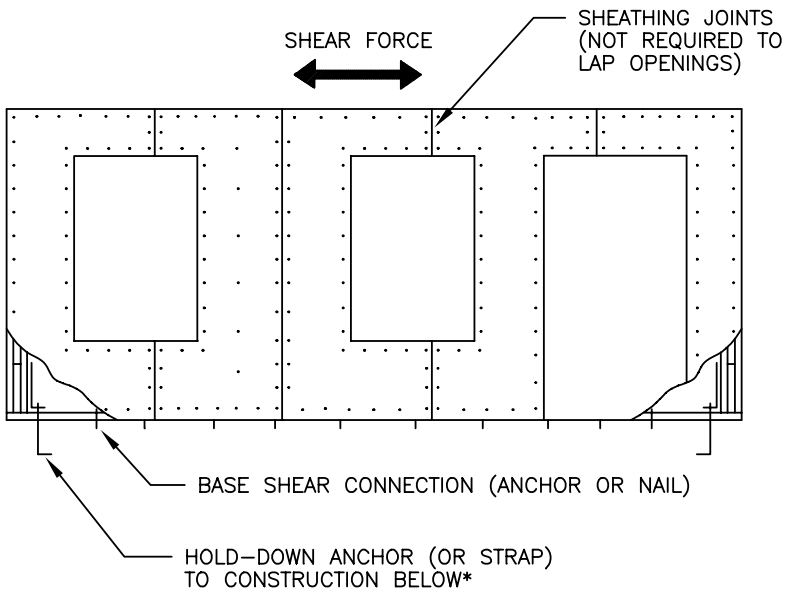
When stiffness is considered, the stiffness of a shear wall segment is assumed to be linearly related to its length (or its total design shear strength). However, the linear relationship is not realistic outside certain limits. For example, stiffness begins to decrease with notable nonlinearly once a shear wall segment decreases below a 4-foot length on an 8-foot-high wall (i.e., aspect ratio of 2 or greater). This does not mean that wall segments shorter than 4 feet in width cannot be used but rather that the effect of relative stiffness in distributing the load needs to be considered. The SSW approach is also less favorable when the wall as a system rather than individual segments (i.e., including sheathed areas above and below openings) may be used to economize on design while meeting required performance (see perforated shear wall design approach below).

As shown in Figure 6.3, it is common either to neglect the contribution of dead load or assume that the dead load on the wall is uniformly distributed as would be the case under gravity loading only. In fact, unless the wall is restrained with an infinitely rigid hold-down device (an impossibility), the uniform dead load distribution will be altered as the wall rotates and deflects upward during the application of shear force (see Figure 6.3b). As a result, depending on the rigidity of the framing system above, the dead load will tend to concentrate more toward the "high points" in the wall line, as the various segments begin to rotate and uplift at their leading edges. Thus, the dead load may be somewhat more effective in offsetting the overturning moment on a shear wall segment than is suggested by the uniform dead load assumption. Unfortunately, this phenomenon involves nonrigid body, nonlinear behavior for which there are no simplified methods of analysis. Therefore, this effect is generally not considered, particularly for walls with specified restraining devices (i.e., hold-downs) that are, by default, generally assumed to be completely rigid–an assumption that is known by testing not to hold true to varying degrees depending on the type of device and its installation.

Basic Perforated Shear Wall (PSW) Design Approach

The *basic perforated shear wall (PSW) design* method is gaining popularity among designers and even earning code recognition. The method, however, is not without controversy in terms of appropriate limits and guidance on use. A perforated shear wall is a wall that is fully sheathed with wood structural panels (i.e., oriented strand board or plywood) and that has openings or "perforations" for windows and doors. The ends of the walls−rather than each individual segment as in the segmented shear wall method−are restrained against overturning. As for the intermediate segments of the wall, they are restrained by conventional or designed framing connections such as those at the base of the wall that transfer the shear force resisted by the wall to the construction below. The capacity of a PSW is determined as the ratio of the strength of a wall with openings to the strength of a wall of the same length without openings. The ratio is calculated by using two empirical equations given in Section 6.5. Figure 6.5 illustrates a perforated shear wall.

FIGURE 6.5 Illustration of a Basic Perforated Shear Wall



The PSW design method requires the least amount of special construction detailing and analysis among the current shear wall design methods. It has been validated in several recent studies in the United States but dates back more than 20 years to research first conducted in Japan (Dolan and Heine, 1997a and b; Dolan and Johnson, 1996a and 1996b; NAHBRC, 1997; NAHBRC, 1998; NAHBRC, 1999; Sugiyama and Matsumoto, 1994; Ni et al., 1998). While it produces the simplest form of an engineered shear wall solution, other methods such as the segmented shear wall design method–all other factors equal–can yield a stronger wall. Conversely, a PSW design with increased sheathing fastening can outperform an SSW with more hold-downs but weaker sheathing fastening. The point is, that for many applications, the PSW method often provides an adequate and more efficient design. Therefore, the PSW method should be considered an option to the SSW method as appropriate.

Enhancements to the PSW Approach

Several options in the form of structural optimizations (i.e., “getting the most from the least”) can enhance the PSW method. One option uses multiple metal straps or ties to restrain each stud, thereby providing a highly redundant and simple method of overturning restraint. Unfortunately, this promising

enhancement has been demonstrated in only one known proof test of the concept (NAHBRC, 1999). It can, however, improve shear wall stiffness and increase capacity beyond that achieved with either the basic PSW method or SSW design approach. Another option, subjected to limited study by the NAHB Research Center, calls for perforated shear walls with metal truss plates at key framing joints (NAHBRC, 1998). To a degree similar to that in the first option, this enhancement increases shear capacity and stiffness without the use of any special hold-downs or restraining devices other than conventional framing connections at the base of the wall (i.e., nails or anchor bolts). Neither of the above options applied dead loads to the tested walls, such application would have improved performance. Unfortunately, the results do not lend themselves to easy duplication by analysis and must be used at their face value as empirical evidence to justify practical design improvements for conditions limited by the tests. Analytic methods are under development to facilitate use of optimization concepts in shear wall design and construction.

In a mechanics-based form of the PSW, analytic assumptions using free-body diagrams and principles of statics can conservatively estimate restraining forces that transfer shear around openings in shear walls based on the assumption that wood-framed shear walls behave as rigid bodies with elastic behavior. As compared to several tests of the perforated shear wall method discussed above, the mechanics-based approach leads to a conservative solution requiring strapping around window openings. In a condition outside the limits for application of the PSW method, a mechanics-based design approach for shear transfer around openings provides a reasonable alternative to traditional SSW design and the newer empirically based PSW design. The added detailing merely takes the form of horizontal strapping and blocking at the top and bottom corners of window openings to transfer the calculated forces derived from free-body diagrams representing the shear wall segments and sheathed areas above and below openings. For more detail, the reader should consult other sources of information on this approach (Diekmann, 1986; ICBO, 1997; ICC, 1999).

6.4.3 Basic Diaphragm Design Approach

As described in Chapter 2 and earlier in this section, horizontal diaphragms are designed by using the analogy of a deep beam laid flatwise. Thus, the shear forces in the diaphragm are calculated as for a beam under a uniform load (refer to Figure 6.4). As is similar to the case of shear walls, the design shear capacity of a horizontal diaphragm is determined by multiplying the diaphragm depth (i.e., depth of the analogous deep beam) by the tabulated unit shear design values found in building codes. The chord forces (in the “flange” of the analogous deep beam) are calculated as a tension force and compression force on opposite sides of the diaphragm. The two forces form a force couple (i.e., moment) that resists the bending action of the diaphragm (refer to Figure 6.1).

To simplify the calculation, it is common practice to assume that the chord forces are resisted by a single chord member serving as the “flange” of the deep beam (i.e., a band joist). At the same time, bending forces internal to the diaphragm are assumed to be resisted entirely by the boundary member or band joist rather than by other members and connections within the diaphragm. In

addition, other parts of the diaphragm boundary (i.e., walls) that also resist the bending tension and compressive forces are not considered. Certainly, a vast majority of residential roof diaphragms that are not considered "engineered" by current diaphragm design standards have exhibited ample capacity in major design events. Thus, the beam analogy used to develop an analytic model for the design of wood-framed horizontal diaphragms has room for improvement that has yet to be explored from an analytic standpoint.

As with shear walls, openings in the diaphragm affect the diaphragm's capacity. However, no empirical design approach accounts for the effect of openings in a horizontal diaphragm as for shear walls (i.e., the PSW method). Therefore, if openings are present, the effective depth of the diaphragm in resisting shear forces must either discount the depth of the opening or be designed for shear transfer around the opening. If it is necessary to transfer shear forces around a large opening in a diaphragm, it is common to perform a mechanics-based analysis of the shear transfer around the opening. The analysis is similar to the previously described method that uses free-body diagrams for the design of shear walls. The reader is referred to other sources for further study of diaphragm design (Ambrose and Vergun, 1987; APA, 1997; Diekmann, 1986).

6.5 Design Guidelines

6.5.1 General Approach

This section outlines methods for designing shear walls (Section 6.5.2) and diaphragms (Section 6.5.3). The two methods of shear wall design are the segmented shear wall (SSW) method and the perforated shear wall (PSW) method. The selection of a method depends on shear loading demand, wall configuration, and the desired simplicity of the final construction. Regardless of design method and resulting LFRS, the first consideration is the amount of lateral load to be resisted by the arrangement of shear walls and diaphragms in a given building. The design loads and basic load combinations in Chapter 3, Table 3.1, are as follows:

- 0.6D + (W or 0.7E) ASD
- 0.9D + (1.5W or 1.0E) LRFD

Earthquake load and wind load are considered separately, with shear walls designed in accordance with more stringent loading conditions.

Lateral building loads should be distributed to the shear walls on a given story by using one of the following methods as deemed appropriate by the designer:

- tributary area approach;
- total shear approach; or
- relative stiffness approach.

These methods were described earlier (see Section 6.4). In the case of the tributary area method, the loads can be immediately assigned to the various shear wall lines based on tributary building areas (exterior surface area for wind loads and building plan area for seismic loads) for the two orthogonal directions of loading (assuming rectangular-shaped buildings and relatively uniform mass distribution for seismic design). In the case of the total shear approach, the load is considered as a "lump sum" for each story for both orthogonal directions of loading. The shear wall construction and total amount of shear wall for each direction of loading and each shear wall line are then determined in accordance with this section to meet the required load as determined by either the tributary area or total shear approach. The designer must be reasonably confident that the distribution of the shear walls and their resistance is reasonably "balanced" with respect to building geometry and the center of the total resultant shear load on each story. As mentioned, both the tributary and total shear approaches have produced many serviceable designs for typical residential buildings, provided that the designer exercises sound judgment.

In the case of the relative stiffness method, the assignment of loads must be based on an assumed relationship describing the relative stiffness of various shear wall lines. Generally, the stiffness of a wood-framed shear wall is assumed to be directly related to the length of the shear wall segments and the unit shear value of the wall construction. For the perforated shear wall method, the relative stiffness of various perforated shear wall lines may be assumed to be directly related to the design strength of the various perforated shear wall lines. Using the principle of moments and a representation of wall racking stiffness, the designer can then identify the center of shear resistance for each story and determine each story's torsional load (due to the offset of the load center from the center of resistance). Finally, the designer superimposes direct shear loads and torsional shear loads to determine the estimated shear loads on each of the shear wall lines.

It is common practice (and required by some building codes) for the torsional load distribution to be used only to add to the direct shear load on one side of the building but not to subtract from the direct shear load on the other side, even though the restriction is not conceptually accurate. Moreover, most seismic design codes require evaluations of the lateral resistance to seismic loads with "artificial" or "accidental" offsets of the estimated center of mass of the building (i.e., imposition of an "accidental" torsional load imbalance). These provisions, when required, are intended to conservatively address uncertainties in the design process that may otherwise go undetected in any given analysis (i.e., building mass is assumed uniform when it actually is not). As an alternative, uncertainties may be more easily accommodated by increasing the shear load by an equivalent amount in effect (i.e., say 10 percent). Indeed, the seismic shear load using the simplified method (see Equation 3.8-1 in Chapter 3) includes a factor that increases the design load by 20 percent and may be considered adequate to address uncertainties in torsional load distribution. However, the simple "20 percent" approach to addressing accidental torsion loads is not explicitly permitted in any current building code. But, for housing, where many redundancies also exist, the "20 percent" rule seems to be a reasonable substitute for a more "exact" analysis of accidental torsion. Of course, it is not a substitute for evaluating and designing for torsion that is expected to occur.

Design Example 6.5 of Section 6.6 elaborates on and demonstrates the use of the methods of load distribution described above. The reader is encouraged to study and critique them. The example contains many concepts and insights that cannot be otherwise conveyed without the benefit of a “real” problem.

6.5.2 Shear Wall Design

6.5.2.1 Shear Wall Design Values (F_s)

This section provides unfactored (ultimate) unit shear values for wood-framed shear wall constructions that use wood structural panels. Other wall constructions and framing methods are included as an additional resource. The unit shear values given here differ from those in the current codes in that they are based explicitly on the ultimate shear capacity as determined through testing. Therefore, the designer is referred to the applicable building code for "code-approved" unit shear values. This guide uses ultimate unit shear capacities as its basis to give the designer an explicit measure of the actual capacity and safety margin (i.e., reserve strength) used in design and to provide for a more consistent safety margin across various shear wall construction options. Accordingly, it is imperative that the values used in this guide are appropriately adjusted in accordance with Sections 6.5.2.2 and 6.5.2.3 to ensure an acceptable safety margin.

Wood Structural Panels (WSP)

Table 6.1 provides unit shear values for walls sheathed with wood structural panels. It should be noted again that these values are estimates of the ultimate unit shear capacity values as determined from several sources (Tissell, 1993; FEMA, 1997; NAHBRC, 1998; NAHBRC, 1999; others). The design unit shear values in today’s building codes have inconsistent safety margins that typically range from 2.5 to 4 after all applicable adjustments (Tissell, 1993; Soltis, Wolfe, and Tuomi, 1983). Therefore, the actual capacity of a shear wall is not explicitly known to the designer using the codes’ allowable unit shear values. Nonetheless, one alleged benefit of using the code-approved design unit shear values is that the values are believed to address drift implicitly by way of a generally conservative safety margin. Even so, shear wall drift is usually not analyzed in residential construction for reasons stated previously.

The values in Table 6.1 and today’s building codes are based primarily on monotonic tests (i.e., tests that use single-direction loading). Recently, the effect of cyclic loading on wood-framed shear wall capacity has generated considerable controversy. However, cyclic testing is apparently not necessary when determining design values for seismic loading of wood-framed shear walls with structural wood panel sheathing. Depending on the cyclic test protocol, the resulting unit shear values can be above or below those obtained from traditional monotonic shear wall test methods (ASTM, 1998a; ASTM, 1998b). In fact, realistic cyclic testing protocols and their associated interpretations were found to be largely in agreement with the results obtained from monotonic testing (Karacabeyli and Ceccotti, 1998). The differences are generally in the range of 10

percent (plus or minus) and thus seem moot given that the seismic response modifier (see Chapter 3) is based on expert opinion (ATC, 1995) and that the actual performance of light-frame homes does not appear to correlate with important parameters in existing seismic design methods (HUD, 1999), among other factors that currently contribute to design uncertainty.

TABLE 6.1 Unfactored (Ultimate) Shear Resistance (plf) for Wood Structural Panel Shear Walls with Framing of Douglas-Fir, Larch, or Southern Pine[1,2]

			Panels Applied Direct to Framing				
				Nail Spacing at Panel Edges (inches)			
Panel Grade	Nominal Panel Thickness (inches)	Minimum Nail Penetration in Framing (inches) (APA, 1998)	Nail Size (common or galvanized box)	6	4	3	2[3]
Structural I	5/16	1-1/4	6d	821	1,122	1,256	1,333
	3/8[4]	1-3/8	8d	833	1,200	1,362	1,711
	7/16[4]	1-3/8	8d	905	1,356	1,497	1,767
	15/32	1-3/8	8d	977	1,539	1,722	1,800
	15/32	1-1/2	10d[5]	1,256	1,701	1,963	2,222

Notes:
[1]Values are average ultimate unit shear capacity for walls sheathed with Structural I wood structural panels and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3. Additional adjustments to the table values should be made in accordance with those sections. For other rated panels (not Structural I), the table values should be multiplied by 0.85.
[2]All panel edges should be backed with 2-inch nominal or wider framing. Panels may be installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8-inch panels installed with the strong axis parallel to studs spaced 24 inches on-center and 12 inches on-center for other conditions and panel thicknesses.
[3]Framing at adjoining panel edges should be 3-inch nominal or wider and nails should be staggered where nails are spaced 2 inches on-center. A double thickness of nominal 2-inch framing is a suitable substitute.
[4]The values for 3/8- and 7/16-inch panels applied directly to framing may be increased to the values shown for 15/32-inch panels, provided that studs are spaced a maximum of 16 inches on-center or the panel is applied with its strong axis across the studs.
[5]Framing at adjoining panel edges should be 3-inch nominal or wider and nails should be staggered where 10d nails penetrating framing by more than 1-5/8 inches are spaced 3 inches or less on-center. A double thickness of 2-inch nominal framing is a suitable substitute.

The unit shear values in Table 6.1 are based on nailed sheathing connections. The use of elastomeric glue to attach wood structural panel sheathing to wood framing members increases the shear capacity of a shear wall by as much as 50 percent or more (White and Dolan, 1993). Similarly, studies using elastomeric construction adhesive manufactured by 3M Corporation have investigated seismic performance (i.e., cyclic loading) and confirm a stiffness increase of about 65 percent and a shear capacity increase of about 45 to 70 percent over sheathing fastened with nails only (Filiatrault and Foschi, 1991). Rigid adhesives may create even greater strength and stiffness increases. The use of adhesives is beneficial in resisting shear loads from wind. Glued shear wall panels are not recommended for use in high-hazard seismic areas because of the brittle failure mode experienced in the wood framing material (i.e., splitting), though at a significantly increased shear load. Gluing shear wall panels is also not recommended by panel manufacturers because of concern with panel buckling that may occur as a result of the interaction of rigid restraints with moisture/temperature expansion and contraction of the panels.

However, construction adhesives are routinely used in floor diaphragm construction to increase the bending stiffness and strength of floors; in-plane (diaphragm) shear is probably affected by an amount similar to that reported above for shear walls.

For unit shear values of wood structural panels applied to cold-formed steel framing, the following references are suggested: *Uniform Building Code* (ICBO,1997); *Standard Building Code* (SBCCI, 1999); and *Shear Wall Values for Light Weight Steel Framing* (AISI, 1996). The unit shear values for cold-formed steel-framed walls in the previous references are consistent with the values used in Table 6.1, including the recommended safety factor or resistance factor. Table 6.2 presents some typical unit shear values for cold-formed steel-framed walls with wood structural panel sheathing fastened with #8 screws. Values for power-driven, knurled pins (similar to deformed shank nails) should be obtained from the manufacturer and the applicable code evaluation reports (NES, Inc., 1997).

TABLE 6.2 Unfactored (Ultimate) Unit Shear Resistance (plf) for Walls with Cold-Formed Steel Framing and Wood Structural Panels[1,2]

Panel Grade	Panel Type and Nominal Thickness (inches)	Minimum Screw Size[3]	Screw Spacing at Panel Edges (inches)[4]			
			6	4	3	2
Structural I	7/16 OSB	#8	700	915	1,275	1,625
	15/32 plywood	#8	780	990	1,465	1,700

Notes:
[1]Values are average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Values apply to 18 gauge (43 mil) and 20 gage (33 mil) steel C-shaped studs with a 1-5/8-inch flange width and 3-1/2- to 5-1/2-inch depth. Studs spaced a maximum of 24 inches on center.
[3]The #8 screws should have a head diameter of no less than 0.29 inches and the screw threads should penetrate the framing so that the threads are fully engaged in the steel.
[4]The spacing of screws in framing members located in the interior of the panels should be no more than 12 inches on-center.

Portland Cement Stucco (PCS)

Ultimate unit shear values for conventional PCS wall construction range from 490 to 1,580 plf based on the ASTM E 72 test protocol and 12 tests conducted by various testing laboratories (Testing Engineers, Inc., 1971; Testing Engineers, Inc., 1970; ICBO, 1969). In general, nailing the metal lath or wire mesh resulted in ultimate unit shear values less than 750 plf, whereas stapling resulted in ultimate unit shear values greater than 750 plf. An ultimate design value of 500 plf is recommended unless specific details of PCS construction are known. A safety factor of 2 provides a conservative allowable design value of about 250 plf. It must be realized that the actual capacity can be as much as five times 250 plf depending on the method of construction, particularly the means of fastening the stucco lath material. Current code-approved allowable design values are typically about 180 plf (SBCCI, 1999; ICBO, 1997). One code requires the values to be further reduced by 50 percent in higher-hazard seismic design areas (ICBO, 1997), although the reduction factor may not necessarily improve performance with respect to the cracking of the stucco finish in seismic events

(HUD, 1999); refer to Chapter 1 and the discussion in Chapter 3 on displacement compatibility under seismic load. It may be more appropriate to use a lower seismic response modifier R than to increase the safety margin in a manner that is not explicit to the designer. In fact, an R factor for PCS wood-framed walls is not explicitly provided in building codes (perhaps an R of 4.5 for "other" wood-framed walls is used) and should probably be in the range of 3 to 4 (without additional increases in the safety factor) since some ductility is provided by the metal lath and its connection to wood framing.

The above values pertain to PCS that is 7/8-inch thick with nail or staple fasteners spaced 6 inches on-center for attaching the metal wire mesh or lath to all framing members. Nails are typically 11 gauge by 1-1/2 inches in length and staples typically have 3/4-inch leg and 7/8-inch crown dimensions. The above unit shear values also apply to stud spacings no greater than 24 inches on-center. Finally, the aspect ratio of stucco wall segments included in a design shear analysis should not be greater than 2 (height/width) according to current building code practice.

Gypsum Wall Board (GWB)

Ultimate capacities in testing 1/2-inch-thick gypsum wall board range from 140 to 300 plf depending on the fastening schedule (Wolfe, 1983; Patton-Mallory, Gutkowski, Soltis, 1984; NAHBRF, date unknown). Allowable or design unit shear values for gypsum wall board sheathing range from 75 to 150 plf in current building codes depending on the construction and fastener spacing. At least one building code requires the values to be reduced by 50 percent in high-hazard seismic design areas (ICBO, 1997). Gypsum wall board is certainly not recommended as the primary seismic bracing for walls, although it does contribute to the structural resistance of buildings in all seismic and wind conditions. It should also be recognized that fastening of interior gypsum board varies in practice and is generally not an 'inspected" system. Table 6.3 provides estimated ultimate unit shear values for gypsum wall board sheathing.

TABLE 6.3 ***Unfactored (Ultimate) Unit Shear Values (plf) for 1/2-Inch-Thick Gypsum Wall Board Sheathing***[1,2]

GWB Thickness	Blocking Condition[3]	Spacing of Framing (inches)	Fastener Spacing at Pane Edges (inches)				
			12	8	7	6	4
1/2 inch	Blocked	16	120	210	250	260	300
	Unblocked	16	80	170	200	220	250
		24	40	120	150	180	220

Notes:
[1]The values represent average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Fasteners should be minimum 1 1/2-inch drywall nails (i.e., 4d cooler) or 1-1/4-inch drywall screws (i.e., #6 size with bugle head) or equivalent with spacing of fasteners and framing members as shown.
[3]"Blocked" refers to panels with all edges fastened to framing members; "unblocked" refers to the condition where the panels are placed horizontally with horizontal joints between panels not fastened to blocking or vertically with the top and bottom edges fastened only at stud locations.

1x4 Wood Let-in Braces and Metal T-braces

Table 6.4 provides values for typical ultimate shear capacities of 1x4 wood let-in braces and metal T-braces. Though not found in current building codes, the values are based on available test data (Wolfe, 1983; NAHBRF, date unknown). Wood let-in braces and metal T-braces are common in conventional residential construction and add to the shear capacity of walls. They are always used in combination with other wall finish materials that also contribute to a wall's shear capacity. The braces are typically attached to the top and bottom plates of walls and at each intermediate stud intersection with two 8d common nails. They are not recommended for the primary lateral resistance of structures in high-hazard seismic or wind design areas. In particular, values of the seismic response modifier R for walls braced in this manner have not been clearly defined for the sake of standardized seismic design guidance.

TABLE 6.4 Unfactored (Ultimate) Shear Resistance (lbs) for 1x4 Wood Let-ins and Metal T-Braces[1,2]

Type of Diagonal Brace	Ultimate Horizontal Shear Capacity (per brace)[3]
1x4 wood let-in brace (8-foot wall height)[4]	600 lbs (tension and compression)
Metal T-brace[5]	1,400 lbs (tension only)

Notes:
[1]Values are average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Values are based on minimum Spruce-Pine-Fir lumber (specific gravity, G = 0.42).
[3]Capacities are based on tests of wall segments that are restrained against overturning.
[4]Installed with two 8d common nails at each stud and plate intersection. Angle of brace should be between 45 and 60 degrees to horizontal.
[5]Installed per manufacturer recommendations and the applicable code evaluation report. Design values may vary depending on manufacturer recommendations, installation requirements, and product attributes.

Other Shear-Resisting Wall Facings

Just about any wall facing, finish, or siding material contributes to a wall's shear resistance qualities. While the total contribution of nonstructural materials to a typical residential building's lateral resistance is often substantial (i.e., nearly 50 percent if interior partition walls are included), current design codes in the United States prohibit considerations of the role of facing, finish, or siding. Some suggestions call for a simple and conservative 10 percent increase (known as the "whole-building interaction factor") to the calculated shear resistance of the shear walls or a similar adjustment to account for the added resistance and whole-building effects not typically considered in design (Griffiths and Wickens, 1996).

Some other types of wall sheathing materials that provide shear resistance include particle board and fiber board. Ultimate unit shear values for fiber board range from 120 plf (6d nail at 6 inches on panel edges with 3/8-inch panel thickness) to 520 plf (10d nail at 2 inches on panel edges with 5/8-inch panel thickness). The designer should consult the relevant building code or manufacturer data for additional information on fiber board and other materials' shear resistance qualities. In one study that conducted tests on various wall assemblies for HUD, fiber board was not recommended for primary shear resistance in high-hazard seismic or wind design areas for the stated reasons of potential durability and cyclic loading concerns (NAHBRF, date unknown).

Combining Wall Bracing Materials

When wall bracing materials (i.e., sheathing) of the same type are used on opposite faces of a wall, the shear values may be considered additive. In high-hazard seismic design conditions, dissimilar materials are generally assumed to be nonadditive. In wind-loading conditions, dissimilar materials may be considered additive for wood structural panels (exterior) with gypsum wall board (interior). Even though let-in brace or metal T-brace (exterior) with gypsum wall board (interior) and fiber board (exterior) with gypsum wall board (interior) are also additive, they are not explicitly recognized as such in current building codes.

When the shear capacity for walls with different facings is determined in accordance with Sections 6.5.2.2 and 6.5.2.3, the designer must take care to apply the appropriate adjustment factors to determine the wall construction's total design racking strength. Most of the adjustment factors in the following sections apply only to wood structural panel sheathing. Therefore, the adjustments in the next section should be made as appropriate before determining combined shear resistance.

6.5.2.2 Shear Wall Design Capacity

The unfactored and unadjusted ultimate unit shear resistance values of wall assemblies should first be determined in accordance with the guidance provided in the previous section for rated facings or structural sheathing materials used on each side of the wall. This section provides methods for determining and adjusting the design unit shear resistance and the shear capacity of a shear wall by using either the perforated shear wall (PSW) approach or segmented shear wall (SSW) approach discussed in Section 6.4.2. The design approaches and other important considerations are illustrated in the design examples of Section 6.6.

Perforated Shear Wall Design Approach

The following equations provide the design shear capacity of a perforated shear wall:

$$F'_s = (F_s)C_{sp}C_{ns}x[\frac{1}{SF} or \phi] \quad \text{(units plf)} \qquad \text{Eq. 6.5-1a}$$

$$F_{psw} = (F'_s)C_{op}C_{dl}x[L] \quad \text{(units lb)} \qquad \text{Eq. 6.5-1b}$$

where,

F_{psw} = the design shear capacity (lb) of the perforated shear wall

F_s = the unfactored (ultimate) and unadjusted unit shear capacity (plf) for each facing of the wall construction; the C_{sp} and C_{ns} adjustment factors apply only to the wood structural panel sheathing F_s values in accordance with Section 6.5.2.1

F'_s = the factored and adjusted design unit shear capacity (plf) for the wall construction

C = the adjustment factors in accordance with Section 6.5.2.3 as applicable
L = the length of the perforated shear wall, which is defined as the distance between the restrained ends of the wall line
$1/SF$ = the safety factor adjustment for use with ASD
ϕ = the resistance factor adjustment for use with LRFD

The PSW method (Equations 6.5-1a and b) has the following limits on its use:

- The value of F_s for the wall construction should not exceed 1,500 plf in accordance with Section 6.5.1.2. The wall must be fully sheathed with wood structural panels on at least one side. Unit shear values of sheathing materials may be combined in accordance with Section 6.5.2.1.
- Full-height wall segments within a perforated shear wall should not exceed an aspect ratio of 4 (height/width) unless that portion of the wall is treated as an opening. (Some codes limit the aspect ratio to 2 or 3.5, but recent testing mentioned earlier has demonstrated otherwise.) The first wall segment on either end of a perforated shear wall must not exceed the aspect ratio limitation.
- The ends of the perforated shear wall must be restrained with hold-down devices sized in accordance with Section 6.5.2.4. Hold-down forces that are transferred from the wall above are additive to the hold-down forces in the wall below. Alternatively, each wall stud may be restrained by using a strap sized to resist an uplift force equivalent to the design unit shear resistance F'_s of the wall, provided that the sheathing area ratio r for the wall is not less than 0.5 (see equations for C_{op} and r in Section 6.5.2.3).
- Top plates must be continuous with a minimum connection capacity at splices with lap joints of 1,000 lb, or as required by the design condition, whichever is greater.
- Bottom plate connections to transfer shear to the construction below (i.e., resist slip) should be designed in accordance with Section 6.5.2.5 and should result in a connection at least equivalent to one 1/2-inch anchor bolt at 6 feet on center or two 16d pneumatic nails 0.131-inch diameter at 24 inches on center for wall constructions with $F_sC_{sp}C_{ns}$ not exceeding 800 plf (ultimate capacity of interior and exterior sheathing). Such connections have been shown to provide an ultimate shear slip capacity of more than 800 plf in typical shear wall framing systems (NAHBRC, 1999); refer to Section 7.3.6 of Chapter 7. For wall constructions with ultimate shear capacities $F_sC_{sp}C_{ns}$ exceeding 800 plf, the base connection must be designed to resist the unit shear load and also provide a design uplift resistance equivalent to the design unit shear load.
- Net wind uplift forces from the roof and other tension forces as a result of structural actions above the wall are transferred through

the wall by using an independent load path. Wind uplift may be resisted with the strapping option above, provided that the straps are sized to transfer the additional load.

Segmented Shear Wall Design Approach

The following equations are used to determine the adjusted and factored shear capacity of a shear wall segment:

$$F'_s = F_s C_{sp} C_{ns} C_{ar} [\frac{1}{SF} \text{ or } \phi] \qquad \text{Eq. 6.5-2a}$$

$$F_{ssw} = F'_s \text{ x}[L_s] \qquad \text{Eq. 6.5-2b}$$

where,

F_{ssw} = the design shear capacity (lb) of a single shear wall segment
F_s = the unfactored (ultimate) and unadjusted unit shear resistance (plf) for the wall construction in accordance with Section 6.5.2.1 for each facing of the wall construction; the C_{sp} and C_{ns} adjustment factors apply only to wood structural panel sheathing F_s values
F'_s = the factored (design) and adjusted unit shear resistance (plf) for the total wall construction
C = the adjustment factors in accordance with Section 6.5.2.3
L_s = the length of a shear wall segment (total width of the sheathing panel(s) in the segment)
1/*SF* = the safety factor adjustment for use with ASD
ϕ = the resistance factor adjustment for use with LRFD

The segmented shear wall design method (Equations 6.5-2a and b) imposes the following limits:

- The aspect ratio of wall segments should not exceed 4 (height/width) as determined by the sheathing dimensions on the wall segment. (Absent an adjustment for the aspect ratio, current codes may restrict the segment aspect ratio to a maximum of 2 or 3.5.)
- The ends of the wall segment should be restrained in accordance with Section 6.5.2.4. Hold-down forces that are transferred from shear wall segments in the wall above are additive to the hold-down forces in the wall below.
- Shear transfer at the base of the wall should be determined in accordance with Section 6.5.2.5.
- Net wind uplift forces from the roof and other tension forces as a result of structural actions above are transferred through the wall by using an independent load path.

For walls with multiple shear wall segments, the design shear resistance for the individual segments may be added to determine the total design shear resistance for the segmented shear wall line. Alternatively, the combined shear

capacity at given amounts of drift may be determined by using the load-deformation equations in Section 6.5.2.6.

6.5.2.3 Shear Capacity Adjustment Factors

Safety and Resistance Factors (SF and ϕ)

Table 6.5 recommends values for safety and resistance factors for shear wall design in residential construction. A safety factor of 2.5 is widely recognized for shear wall design, although the range varies substantially in current code-approved unit shear design values for wood-framed walls (i.e., the range is 2 to more than 4). In addition, a safety factor of 2 is commonly used for wind design. The 1.5 safety factor for ancillary buildings is commensurate with lower risk but may not be a recognized practice in current building codes. A safety factor of 2 has been historically applied or recommended for residential dwelling design (HUD, 1967; MPS, 1958; HUD, 1999). It is also more conservative than safety factor adjustments typically used in the design of other properties with wood members and other materials.

TABLE 6.5 Minimum Recommended Safety and Resistance Factors for Residential Shear Wall Design

Type of Construction		Safety Factor (ASD)	Resistance Factor (LRFD)
Detached garages and ancillary buildings not for human habitation		1.5	1.0
Single-family houses, townhouses, and multifamily low-rise buildings (apartments)	Seismic	2.5	0.55
	Wind	2.0	0.7

Species Adjustment Factor (C_{sp})

The ultimate unit shear values for wood structural panels in Table 6.1 apply to lumber species with a specific gravity (density), G, greater than or equal to 0.5. Table 6.6 presents specific gravity values for common species of lumber used for wall framing. For G < 0.5, the following value of C_{sp} should be used to adjust values in Table 6.1 only (APA, 1998):

$$C_{sp} = [1-(0.5-G)] \le 1.0 \qquad \text{Eq. 6.5-3}$$

TABLE 6.6 Specific Gravity Values (Average) for Common Species of Framing Lumber

Lumber Species	Specific Gravity, G
Southern Yellow Pine (SYP)	0.55
Douglas Fir-Larch (DF-L)	0.50
Hem-Fir (HF)	0.43
Spruce-Pine-Fir (SPF)	0.42

Nail Size Adjustment Factor (C_{ns})

The ultimate unit shear capacities in Table 6.1 are based on the use of common nails. For other nail types and corresponding nominal sizes, the C_{ns} adjustment factors in Table 6.7 should be used to adjust the values in Table 6.1. Nails should penetrate framing members a minimum of 10D, where D is the diameter of the nail.

TABLE 6.7 Values of C_{ns} for Various Nail Sizes and Types[1]

Nominal Nail Size (penny weight)	Nail Length (inches)	Nail Type					
		Common[2]	Box[3]	Pneumatic (by diameter in inches)			
				0.092	0.113	0.131	0.148
6d	1-7/8 to 2	1.0	0.8	0.9	1.0	N/A[4]	N/A[4]
8d	2-3/8 to 2-1/2	1.0	0.8	0.5	0.75	1.0	N/A[4]
10d	3	1.0	0.8	N/A[4]	N/A[4]	0.8	1.0

Notes:
[1]The values of C_{ns} are based on ratios of the single shear nail values in NER-272 (NES, Inc., 1997) and the NDS (AF&PA, 1997) and are applicable only to wood structural panel sheathing on wood-framed walls in accordance with Table 6.1.
[2]Common nail diameters are as follows: 6d (0.113 inch), 8d (0.131 inch), and 10d (0.148 inch).
[3]Box nail diameters are as follows: 6d (0.099 inch), 8d (0.113 inch), and 10d (0.128 inch).
[4]Diameter not applicable to nominal nail size. Nail size, diameter, and length should be verified with the manufacturer.

Opening Adjustment Factor (C_{op})

The following equation for C_{op} applies only to the perforated shear wall method in accordance with Equation 6.5-1b of Section 6.5.2.2:

$$C_{op} = r/(3-2r) \qquad \text{Eq. 6.5-4}$$

where,

$r = 1/(1 + \alpha/\beta)$ = sheathing area ratio (dimensionless)

$\alpha = \Sigma A_o / (H \times L)$ = ratio of area of all openings ΣA_o to total wall area, H x L (dimensionless)

$\beta = \Sigma L_i / L$ = ratio of length of wall with full-height sheathing ΣL_i to the total wall length L of the perforated shear wall (dimensionless)

Dead Load Adjustment Factor (C_{dl})

The C_{dl} factor applies to the perforated shear wall method only (Equation 6.5-1b). The presence of a dead load on a perforated shear has the effect of increasing shear capacity (Ni et al., 1998). The increase is 15 percent for a uniform dead load of 300 plf or more applied to the top of the wall framing. The dead load should be decreased by wind uplift and factored in accordance with the lateral design load combinations of Chapter 3. The C_{dl} adjustment factor is determined as follows and should not exceed 1.15:

$$C_{dl} = 1 + 0.15\left(\frac{w_D}{300}\right) \leq 1.15 \qquad \text{Eq 6.5-5}$$

where,

w_D = the net uniform dead load supported at the top of the perforated shear wall (plf) with consideration of wind uplift and factoring in accordance with load combinations of Chapter 3.

Aspect Ratio Adjustment Factor (C_{ar})

The following C_{ar} adjustment factor applies only to the segmented shear wall design method for adjusting the shear resistance of interior and exterior sheathing in accordance with Equation 6.5-2a of Section 6.5.2.2:

$$C_{ar} = \frac{1}{\sqrt{0.5(a)}} \text{ for } 2.0 \leq a \leq 4.0 \qquad \text{Eq 6.5-6}$$

$C_{ar} = 1.0$ for $a < 2.0$

where,

a is the aspect ratio (height/width) of the sheathed shear wall segment.

6.5.2.4 Overturning Restraint

Section 6.3 and Figure 6.3 address overturning restraint of shear walls in conceptual terms. In practice, the two generally recognized approaches to providing overturning restraint call for

- the evaluation of equilibrium of forces on a *restrained* shear wall segment using principles of engineering mechanics; or
- the evaluation of *unrestrained* shear walls considering nonuniform dead load distribution at the top of the wall with restraint provided by various connections (i.e., sheathing, wall bottom plate, corner framing, etc.).

The first method applies to restrained shear wall segments in both the perforated and segmented shear wall methods. The first segment on each end of a perforated shear wall is restrained in one direction of loading. Therefore, the overturning forces on that segment are analyzed in the same manner as for a segmented shear wall. The second method listed above is a valid and conceptually realistic method of analyzing the restraint of typical residential wall constructions, but it has not yet fully matured. Further, the method's load path (i.e., distribution of uplift forces to various connections with inelastic properties) is perhaps beyond the practical limits of a designer's intuition. Rather than presume a methodology based on limited testing (see Section 6.3), this guide does not suggest guidelines for the second approach. However, the second method is worth consideration by a designer when attempting to understand the performance of conventional,

"nonengineered" residential construction. Mechanics-based methods to assist in the more complicated design approach are under development.

Using basic mechanics as shown in Figure 6.6, the following equation for the chord tension and compression forces are determined by summing moments about the bottom compression or tension side of a restrained shear wall segment:

$$\sum M_C = 0$$

$$F'_s\,(d)(h) - T\;(x) - D_W\left(\tfrac{1}{2}d\right) - (w_D)(d)\left(\tfrac{1}{2}d\right) = 0$$

$$T = \left(\frac{d}{x}\right)\left(F'_s\,h - \tfrac{1}{2}D_W - \tfrac{1}{2}(w_D)(d)\right) + t \qquad \text{Eq. 6.5-7a}$$

$$\sum M_T = 0$$

$$C = \left(\frac{d}{x}\right)\left(F'_s\,h + \tfrac{1}{2}D_W + \tfrac{1}{2}(w_D)(d)\right) + c \qquad \text{Eq. 6.5-7b}$$

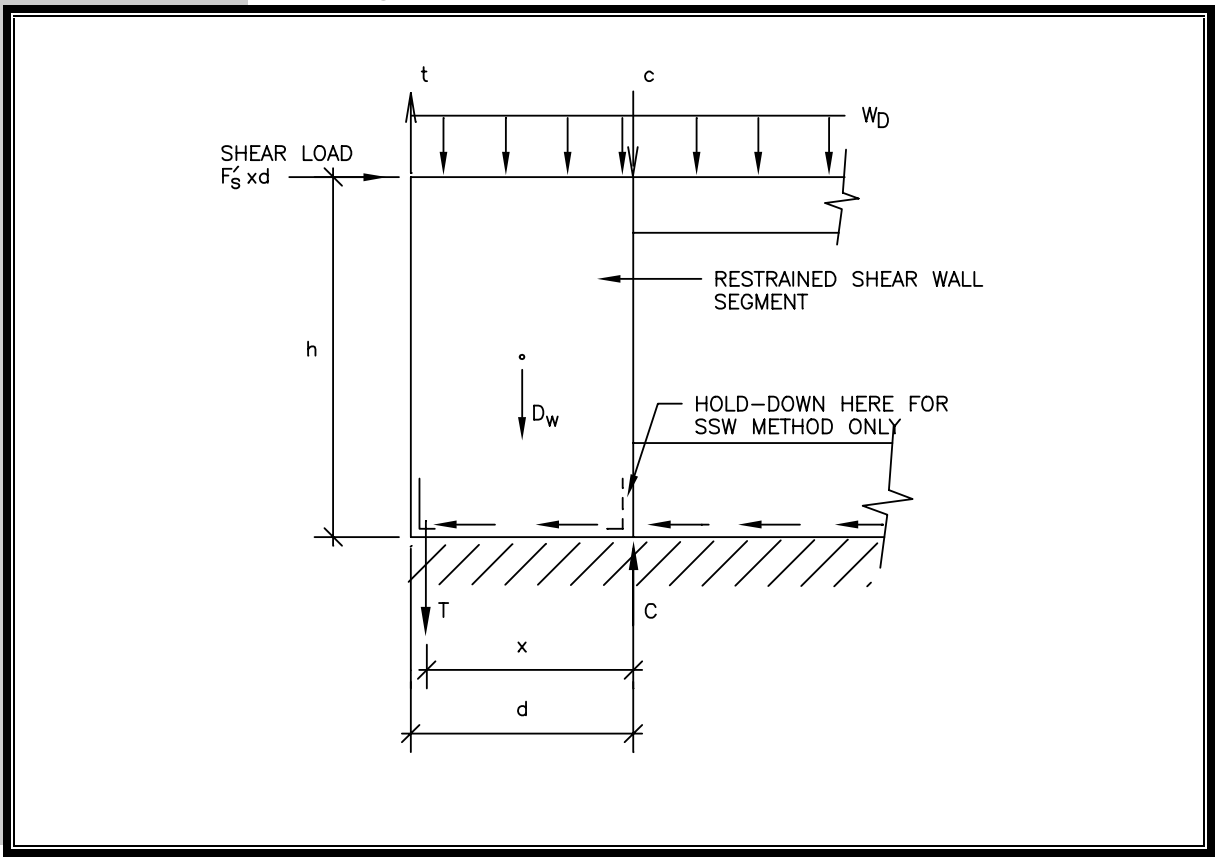
where,

- T = the tension force on the hold-down device (lb)
- d = the width of the restrained shear wall segment (ft); for segments greater than 4 ft in width, use d = 4 ft.
- x = the distance between the hold-down device and the compression edge of the restrained shear wall segment (ft); for segments greater than 4 ft in width, use x = 4 ft plus or minus the bracket offset dimension, if any
- F'_s = the design unit shear capacity (plf) determined in accordance with Equation 6.5-2a of Section 6.5.2.2 (for both the PSW and SSW methods)
- h = the height of the wall (ft)
- D_w = the dead load of the shear wall segment (lb); dead load must be factored and wind uplift considered in accordance with the load combinations of Chapter 3.
- w_D = the uniform dead load supported by the shear wall segment (plf); dead load must be factored and wind uplift considered in accordance with the load combinations of Chapter 3.
- t = the tension load transferred through a hold-down device, if any, restraining a wall above (lb); if there is no tension load, t = 0
- c = the compression load transferred from wall segments above, if any (lb); this load may be distributed by horizontal structural elements above the wall (i.e., not a concentrated load); if there is not compression load, c = 0.

The 4-foot-width limit for *d* and *x* is imposed on the analysis of overturning forces as presented above because longer shear wall lengths mean that the contribution of the additional dead load cannot be rigidly transferred

through deep bending action of the wall to have a full effect on the uplift forces occurring at the end of the segment, particularly when it is rigidly restrained from uplifting. This effect also depends on the stiffness of the construction above the wall that "delivers" and distributes the load at the top of the wall. The assumptions necessary to include the restraining effects of dead load is no trivial matter and, for that reason, it is common practice to not include any beneficial effect of dead load in the overturning force analysis of individual shear wall segments.

FIGURE 6.6 ***Evaluation of Overturning Forces on a Restrained Shear Wall Segment***



For a more simplified analysis of overturning forces, the effect of dead load may be neglected and the chord forces determined as follows using the symbols defined as before:

$$T = C = \left(\frac{d}{x}\right) F'_s \, h \qquad \text{Eq. 6.5-7c}$$

Any tension or compression force transferred from shear wall overturning forces originating above the wall under consideration must be added to the result of Equation 6.5-7c as appropriate. It is also assumed that any net wind uplift force is resisted by a separate load path (i.e., wind uplift straps are used in addition to overturning or hold-down devices).

For walls not rigidly restrained, the initiation of overturning uplift at the end stud (i.e., chord) shifts an increasing amount of the dead load supported by the wall toward the leading edge. Thus, walls restrained with more flexible hold-down devices or without such devices benefit from increased amounts of offsetting dead load as well as from the ability of wood framing and connections to disperse some of the forces that concentrate in the region of a rigid hold-down device. However, if the bottom plate is rigidly anchored, flexibility in the hold-down device can impose undesirable cross-grain bending forces on the plate due to uplift forces transferred through the sheathing fasteners to the edge of the bottom plate. Further, the sheathing nails in the region of the bottom plate anchor experience greater load and may initiate failure of the wall through an "unzipping" effect.

The proper detailing to balance localized stiffness effects for more even force transfer is obviously a matter of designer judgment. It is mentioned here to emphasize the importance of detailing in wood-framed construction. In particular, wood framing has the innate ability to distribute loads, although weaknesses can develop from seemingly insignificant details. The concern noted above has been attributed to actual problems (i.e., bottom plate splitting) only in severe seismic events and in relatively heavily loaded shear walls. For this reason, it is now common to require larger washers on bottom plate anchor bolts, such as a 2- to 3-inch-square by 1/4-inch-thick plate washer, to prevent the development of cross-grain tension forces in bottom plates in high-hazard seismic regions. The development of high cross-grain tension stresses poses less concern when nails are used to fasten the bottom plate and are located in pairs or staggered on both sides of the wood plate. Thus, the two connection options above represent different approaches. The first, using the plate washers, maintains a rigid connection throughout the wall to prevent cross grain tension in the bottom plate. The second, using nails, is a more "flexible" connection that prevents concentrated cross-grain bending forces from developing. With sufficient capacity provided, the nailing approach may yield a more "ductile" system. Unfortunately, these intricate detailing issues are not accommodated in the single seismic response modifier used for wood-framed shear walls or the provisions of any existing code. These aspects of design are not easily "quantified" and are considered matters of qualitative engineering judgment.

Finally, it is important to recognize that the hold-down must be attached to a vertical wall framing member (i.e., a stud) that receives the wood structural panel edge nailing. If not, the hold-down will not be fully effective (i.e., the overturning forces must be "delivered" to the hold-down through the sheathing panel edge nailing). In addition, the method of deriving hold-down capacity ratings may vary from bracket to bracket and manufacturer to manufacturer. For some brackets, the rated capacity may be based on tests of the bracket itself that do not represent its use in an assembly (i.e., as attached to a wood member). Many hold-down brackets transfer tension through an eccentric load path that creates an end moment on the vertical framing member to which it is attached. Therefore, there may be several design considerations in specifying an appropriate hold-down device that go beyond simply selecting a device with a sufficient rated capacity from manufacturer literature. In response to these issues, some local codes may require certain reductions to or verification of rated hold-down capacities.

6.5.2.5 Shear Transfer (Sliding)

The sliding shear at the base of a shear wall is equivalent to the shear load input to the wall. To ensure that the sliding shear force transfer is balanced with the shear capacity of the wall, the connections at the base of the wall are usually designed to transfer the design unit shear capacity F'_s of the shear wall. Generally, the connections used to resist sliding shear include anchor bolts (fastening to concrete) and nails (fastening to wood framing). Metal plate connectors may also be used (consult manufacturer literature). In what is a conservative decision, frictional resistance and "pinching" effects usually go ignored. However, if friction is considered, a friction coefficient of 0.3 may be multiplied by the dead load normal to the slippage plane to determine a nominal resistance provided by friction.

As a modification to the above rule, if the bottom plate is continuous in a perforated shear wall, the sliding shear resistance is the capacity of the perforated shear wall F_{psw}. If the bottom plate is not continuous, then the sliding shear should be designed to resist the design unit shear capacity of the wall construction F'_s as discussed above. Similarly, if the restrained shear wall segments in a segmented shear wall line are connected to a continuous bottom plate extending between shear wall segments, then the sliding shear can be distributed along the entire length of the bottom plate. For example, if two 4-foot shear wall segments are located in a wall 12 feet long with a continuous bottom plate, then the unit sliding shear resistance required at the bottom plate anchorage is (8 ft)(F'_s)/(12 ft) or 2/3(F'_s). This is similar to the mechanism by which a unit shear load is transferred from a horizontal diaphragm to the wall top plate and then into the shear wall segments through a collector (i.e., top plate). Chapter 7 addresses design of the above types of shear connections.

6.5.2.6 Shear Wall Stiffness and Drift

The methods for predicting shear wall stiffness or drift in this section are based on idealized conditions representative solely of the testing conditions to which the equations are related. The conditions do not account for the many factors that may decrease the actual drift of a shear wall in its final construction. As mentioned, shear wall drift is generally overestimated in comparison with actual behavior in a completed structure (see Section 6.2 on whole-building tests). The degree of overprediction may reach a factor of 2 at design load conditions. At capacity, the error may not be as large because some nonstructural components may be past their yield point.

At the same time, drift analysis may not consider the factors that also increase drift, such as deformation characteristics of the hold-down hardware (for hardware that is less stiff than that typically used in testing), lumber shrinkage (i.e., causing time-delayed slack in joints), lumber compression under heavy shear wall compression chord load, and construction tolerances. Therefore, the results of a drift analysis should be considered as a guide to engineering judgment, not an exact prediction of drift.

The load-drift equations in this section may be solved to yield shear wall resistance for a given amount of shear wall drift. In this manner, a series of shear wall segments or even perforated shear walls embedded within a given wall line may be combined to determine an overall load-drift relationship for the entire wall line. The load-drift relationships are based on the nonlinear behavior of wood-framed shear walls and provide a reasonably accurate means of determining the behavior of walls of various configurations. The relationship may also be used for determining the relative stiffness of shear wall lines in conjunction with the relative stiffness method of distributing lateral building loads and for considering torsional behavior of a building with a nonsymmetrical shear wall layout in stiffness and in geometry. The approach is fairly straightforward and is left to the reader for experimentation.

Perforated Shear Wall Load-Drift Relationship

The load-drift equation below is based on several perforated shear wall tests already discussed in this chapter. It provides a nonlinear load-drift relationship up to the ultimate capacity of the perforated shear wall as determined in Section 6.5.2.2. When considering shear wall load-drift behavior in an actual building, the reader is reminded of the aforementioned accuracy issues; however, accuracy relative to the test data is reasonable (i.e., plus or minus 1/2-inch at capacity).

$$\Delta = 1.8\left(\frac{0.5}{G}\right)\left(\frac{1}{\sqrt{r}}\right)\left(\frac{V_d}{F_{PSW,ULT}}\right)^{2.8}\left[\frac{h}{8}\right] \text{ (inches)} \qquad \text{Eq. 6.5-8}$$

where,

Δ = the shear wall drift (in) at shear load demand, V_d (lb)
G = the specific gravity of framing lumber (see Table 6.6)
r = the sheathing area ratio (see Section 6.5.2.3, C_{op})
V_d = the shear load demand (lb) on the perforated shear wall; the value of V_d is set at any unit shear demand less than or equal to $F_{psw,ult}$ while the value of V_d should be set to the design shear load when checking drift at design load conditions
$F_{psw,ult}$ = the unfactored (ultimate) shear capacity (lb) for the perforated shear wall (i.e., F_{psw} x *SF* or F_{psw}/ϕ for ASD and LRFD, respectively)
h = the height of wall (ft)

Segmented Shear Wall Load-Drift Relationship

APA Semiempirical Load-Drift Equation

Several codes and industry design guidelines specify a deflection equation for shear walls that includes a multipart estimate of various factors' contribution to shear wall deflection (ICBO, 1997; ICC, 1999, APA, 1997). The approach relies on a mix of mechanics-based principles and empirical modifications. The principles and modifications are not repeated here because the APA method of

drift prediction is considered no more reliable than that presented next. In addition, the equation is complex relative to the ability to predict drift accurately. It also requires adjustment factors, such as a nail-slip factor, that can only be determined by testing.

Empirical, Nonlinear Load-Drift Equation

Drift in a wood structural panel shear wall segment may be approximated in accordance with the following equation:

$$\Delta = 2.2\left(\frac{0.5}{G}\right)\sqrt[4]{a}\left(\frac{V_d}{F_{SSW,ULT}}\right)^{2.8}\left[\frac{h}{8}\right] \text{ (in)} \qquad \text{Eq. 6.5-9}$$

where,

Δ = the shear wall drift (in) at load V_d (lb)
G = the specific gravity of framing lumber
a = the shear wall segment aspect ratio (height/width) for aspect ratios from 4 to 1; a value of 1 shall be used for shear wall segments with width (length) greater than height
V_d = the shear load demand (lb) on the wall; the value of V_d is set at any unit shear demand less than or equal to $F_{ssw,ult}$ while the value of V_d should be set to the design load when checking drift at design load conditions
$F_{ssw,ult}$ = the unfactored (ultimate) shear capacity (lb) of the shear wall segment (i.e., F_{ssw} x *SF* or F_{ssw}/ϕ for ASD and LRFD, respectively)
h = the height of wall (ft)

The above equation is based on several tests of shear wall segments with aspect ratios ranging from 4:1 to 1:5.

6.5.2.7 Portal Frames

In situations with little space to include sufficient shear walls to meet required loading conditions, the designer must turn to alternatives. An example is a garage opening supporting a two-story home on a narrow lot such that other wall openings for windows and an entrance door leaves little room for shear walls. One option is to consider torsion and the distribution of lateral loads in accordance with the relative stiffness method. Another possibility is the use of a portal frame.

Portal frames may be simple, specialized framing details that can be assembled on site. They use fastening details, metal connector hardware, and sheathing to form a wooden moment frame and, in many cases, perform adequately. Various configurations of portal frames have undergone testing and provide data and details on which the designer can base a design (NAHBRC, 1998; APA, 1994). The ultimate shear capacity of portal frames ranges from 2,400 to more than 6,000 pounds depending on the complexity and strength of the construction details. A simple detail involves extending a garage header so that it

is end-nailed to a full-height corner stud, strapping the header to the jamb studs at the portal opening, attaching sheathing with a standard nailing schedule, and anchoring the portal frame with typical perforated shear wall requirements. The system has an ultimate shear capacity of about 3,400 pounds that, with a safety factor of 2 to 2.5, provides a simple solution for many portal frame applications for residential construction in high-hazard seismic or wind regions. Several manufacturers offer preengineered portal frame and shear wall elements that can be ordered to custom requirements or standard conditions.

6.5.3 Diaphragm Design

6.5.3.1 Diaphragm Design Values

Depending on the location and number of supporting shear wall lines, the shear and moments on a diaphragm are determined by using the analogy of a simply supported or continuous span beam. The designer uses the shear load on the diaphragm per unit width of the diaphragm (i.e., floor or roof) to select a combination of sheathing and fastening from a table of allowable horizontal diaphragm unit shear values found in U.S. building codes. Similar to those for shear walls, unit shear values for diaphragms vary according to sheathing thickness and nailing schedules, among other factors. Table 6.8 presents several of the more common floor and roof constructions used in residential construction as well as their allowable diaphragm resistance values. The values include a safety factor for ASD and therefore require no additional factoring. The aspect ratio of a diaphragm should be no greater than 4 (length/width) in accordance with current building code limits. In addition, the sheathing attachment in floor diaphragms is often supplemented with glue or construction adhesive. The increase in unit shear capacity of vertical diaphragms (i.e. shear walls) was discussed in Section 6.5.2.1 in association with Table 6.1. A similar increase to the unit shear capacity of floor diaphragms can be expected, not to mention increased stiffness when the floor sheathing is glued and nailed.

TABLE 6.8 ***Horizontal Diaphragm ASD Shear Values (plf) for Unblocked Roof and Floor Construction Using Douglas Fir or Southern Pine Framing***[1,2,3,4]

Panel Type and Application	Nominal Panel Thickness (inches)	Common Nail Size	Design Shear Value (plf)
Structural I (Roof)	5/16	6d	165
	3/8	8d	185
	15/32	10d	285
APA Sturd-I-Floor (Floor) and Rated Sheathing	7/16	8d	230
	15/32	8d	240
	19/32	10d	285

Notes:
[1]Minimum framing member thickness is 1-1/2 inches.
[2]Nails spaced at 6 inches on-center at supported panel edges and at the perimeter of the diaphragm. Nails spaced at 12 inches on-center on other framing members spaced a maximum of 24 inches on-center.
[3]"Unblocked" means that sheathing joints perpendicular to framing members are not fastened to blocking.
[4]Apply C_{sp} and C_{ns} adjustment factors to table values as appropriate (see Section 6.5.2.3 for adjustment factor values).

6.5.3.2 Diaphragm Design

As noted, diaphragms are designed in accordance with simple beam equations. To determine the shear load on a simply supported diaphragm (i.e., diaphragm supported by shear walls at each side), the designer uses the following equation to calculate the unit shear force to be resisted by the diaphragm sheathing:

$$V_{max} = \frac{1}{2} wl \qquad \text{Eq. 6.5-10a}$$

$$v_{max} = \frac{V_{max}}{d} \qquad \text{Eq. 6.5-10b}$$

where,

V_{max} = the maximum shear load on the diaphragm (plf)
w = the tributary uniform load (plf) applied to the diaphragm resulting from seismic or wind loading
l = the length of the diaphragm perpendicular to the direction of the load (ft)
v_{max} = the unit shear across the diaphragm in the direction of the load (plf)
d = the depth or width of the diaphragm in the direction of the load (ft)

The following equations are used to determine the theoretical chord tension and compression forces on a simply supported diaphragm as described above:

$$M_{max} = \frac{1}{8} wl^2 \qquad \text{Eq. 6.5-11a}$$

$$T_{max} = C_{max} = \frac{M_{max}}{d} \qquad \text{Eq. 6.5-11b}$$

where,

M_{max} = the bending moment on the diaphragm (ft-lb)
w = the tributary uniform load (plf) applied to the diaphragm resulting from seismic or wind loading
l = the length of the diaphragm perpendicular to the direction of the load (ft)
T_{max} = the maximum chord tension force (lb)
C_{max} = the maximum chord compression force (lb)
d = the depth or width of the diaphragm in the direction of the load (ft)

If the diaphragm is not simply supported at its ends, the designer uses appropriate beam equations (see Appendix A) in a manner similar to that above to determine the shear and moment on the diaphragm. The calculations to determine the unit shear in the diaphragm and the tension and compression in the chords are

also similar to those given above. It should be noted that the maximum chord forces occur at the location of the maximum moment. For a simply supported diaphragm, the maximum chord forces occur at mid-span between the perimeter shear walls. Thus, chord requirements may vary depending on location and magnitude of the bending moment on the diaphragm. Similarly, shear forces on a simply supported diaphragm are highest near the perimeter shear walls (i.e., reactions). Therefore, nailing requirements for diaphragms may be adjusted depending on the variation of the shear force in interior regions of the diaphragm. Generally, these variations are not critical in small residential structures such that fastening schedules can remain constant throughout the entire diaphragm. If there are openings in the horizontal diaphragm, the width of the opening dimension is usually discounted from the width *d* of the diaphragm when determining the unit shear load on the diaphragm.

6.5.3.3 Shear Transfer (Sliding)

The shear forces in the diaphragm must be adequately transferred to the supporting shear walls. For typical residential roof diaphragms, conventional roof framing connections are often sufficient to transfer the small sliding shear forces to the shear walls (unless heavy roof coverings are used in high-hazard seismic areas or steep roof slopes are used in high-hazard wind regions). The transfer of shear forces from floor diaphragms to shear walls may also be handled by conventional nailed connections between the floor boundary member (i.e., a band joist or end joist that is attached to the floor diaphragm sheathing) and the wall framing below. In heavily loaded conditions, metal shear plates may supplement the connections. The simple rule to follow for these connections is that the shear force in from the diaphragm must equal the shear force out to the supporting wall. Floors supported on a foundation wall are usually connected to a wood sill plate bolted to the foundation wall; however, the floor joist and/or the band joist may be directly connected to the foundation wall. Chapter 7 addresses the design of these shear connections.

6.5.3.4 Diaphragm Stiffness

Diaphragm stiffness may be calculated by using semi-empirical methods based on principles of mechanics. The equations are found in most modern building codes and industry guidelines (APA, 1997; ICBO, 1997; ICC, 1999). For typical residential construction, however, the calculation of diaphragm deflection is almost never necessary and rarely performed. Therefore, the equations and their empirical adjustment factors are not repeated here. Nonetheless, the designer who attempts diaphragm deflection or stiffness calculations is cautioned regarding the same accuracy concerns mentioned for shear wall drift calculations. The stiffness of floor and roof diaphragms is highly dependent on the final construction, including interior finishes (see Section 6.2 on whole-building tests).

6.6 Design Examples

EXAMPLE 6.1 Segmented Shear Wall Design

Given

The segmented shear wall line, as shown in the figure below, has the following dimensions:

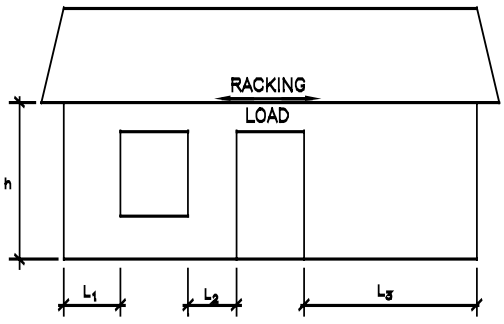
h = 8 ft
L_1 = 3 ft
L_2 = 2 ft
L_3 = 8 ft

Wall construction:

- Exterior sheathing is 7/16-inch-thick OSB with 8d pneumatic nails (0.113 inch diameter by 2 3/8 inches long) spaced 6 inches on center on panel edges and 12 inches on center in panel field
- Interior sheathing is 1/2-inch-thick gypsum wall board with #6 screws at 12 inches on center
- Framing lumber is Spruce-Pine-Fir, Stud grade (specific gravity, G = 0.42); studs are spaced at 16 inches on center.

Loading condition (assumed for illustration)

Wind shear load on wall line = 3,000 lb
Seismic shear load on wall line = 1,000 lb



Find

1. Design capacity of the segmented shear wall line for wind and seismic shear resistance.
2. Base shear connection requirements.
3. Chord tension and compression forces.
4. Load-drift behavior of the segmented shear wall line and estimated drift at design load conditions.

Solution

1. Determine the factored and adjusted (design) shear capacities for the wall segments and the total wall line (Section 6.5.2).

$F_{s,ext}$ = 905 plf OSB sheathing (Table 6.1)
$F_{s,int}$ = 80 plf GWB sheathing (Table 6.3)

The design shear capacity of the wall construction is determined as follows for each segment (Sections 6.5.2.1 and 6.5.2.2):

$F'_s = F'_{s,ext} + F'_{s,int}$
$F'_s = F_{s,ext} C_{sp} C_{ns} C_{ar} [1/SF] + F_{s,int} C_{ar} [1/SF]$

C_{sp} = [1-(0.5-0.42)] = 0.92 (Section 6.5.2.3)
C_{ns} = 0.75 (Table 6.7)
SF = 2.0 (wind) or 2.5 (seismic) (Table 6.5)

Segment 1

a = h/L_1 = (8 ft)/(3 ft) = 2.67 (segment aspect ratio)
C_{ar} = 1/sqrt(0.5(a)) = 0.87 (Section 6.5.2.3)

For wind design

$F'_{s,1,wind}$ = (905 plf)(0.92)(0.75)(0.87)(1/2.0) + (80 plf)(0.87)(1/2.0)
= 272 plf + 35 plf = 307 plf
$F_{ssw,1,wind}$ = $F'_s(L_1)$ = (307 plf)(3 ft) = 921 lb

For seismic design

$F'_{s,1,seismic}$ = (905 plf)(0.92)(0.75)(0.87)(1/2.5) + 0 = 218 plf
$F_{ssw,1,seismic}$ = (218 plf)(3 ft) = 654 lb

Segment 2

a = h/L_2 = (8 ft)/(2 ft) = 4
C_{ar} = 1/sqrt(0.5(a)) = 0.71

For wind design

$F'_{s,2,wind}$ = (905 plf)(0.92)(0.75)(0.71)(1/2.0) + (80 plf)(0.71)(1/2.0)
= 222 plf + 28 plf = 250 plf
$F_{ssw,2,wind}$ = (250 plf)(2 ft) = 500 lb

For seismic design

$F'_{s,2,seismic}$ = (905 plf)(0.92)(0.75)(0.71)(1/2.5) + 0 = 178 plf
$F_{ssw,2,seismic}$ = (178 plf)(2 ft) = 356 lb

Segment 3

$a = h/L_3 = (8\text{ ft})/(8\text{ ft}) = 1$
$C_{ar} = 1.0$ (for $a < 2$)

For wind design

$F'_{s,3,wind} = (905\text{ plf})(0.92)(0.75)(1.0)(1/2.0) + (80\text{ plf})(1.0)(1/2.0)$
$= 312\text{ plf} + 40\text{ plf} = 352\text{ plf}$
$F_{ssw,3,wind} = (352\text{ plf})(8\text{ ft}) = 2{,}816\text{ lb}$

For seismic design

$F'_{s,3,seismic} = (905\text{ plf})(0.92)(0.75)(1.0)(1/2.5) + 0 = 250\text{ plf}$
$F_{ssw,3,seismic} = (250\text{ plf})(8\text{ ft}) = 2{,}000\text{ lb}$

Total for wall line

$F_{ssw,total,wind} = 921\text{ lb} + 500\text{ lb} + 2{,}816\text{ lb} = 4{,}237\text{ lb}$
$F_{ssw,total,seismic} = 654\text{ lb} + 356\text{ lb} + 2{,}000\text{ lb} = 3{,}010\text{ lb}$

2. Determine base shear connection requirements to transfer shear load to the foundation or floor construction below the wall

The wall bottom plate to the left of the door opening is considered to be continuous and therefore acts as a distributor of the shear load resisted by Segments 1 and 2. The uniform shear connection load on the bottom plate to the left of the opening is determined as follows:

Bottom plate length $= 3\text{ ft} + 3\text{ ft} + 2\text{ ft} = 8\text{ ft}$

Base shear resistance required (wind) $= (F_{ssw,1,wind} + F_{ssw,2,wind})/(\text{plate length})$
$= (921\text{ lb} + 500\text{ lb})/(8\text{ ft}) = 178\text{ plf}$

Base shear resistance required (seismic) $= (F_{ssw,1,seismic} + F_{ssw,2,seismic})/(\text{plate length})$
$= (654\text{ lb} + 356\text{ lb})/(8\text{ ft}) = 127\text{ plf}$

For the wall bottom plate to the right of the door opening, the base shear connection is equivalent to $F'_{s,3,wind} = 352$ plf or $F'_{s,3,seismic} = 250$ plf for wind and seismic design respectively.

Normally, this connection is achieved by use of nailed or bolted bottom plate fastenings. Refer to Chapter 7 and Section 7.3.6 for information on designing these connections.

Notes:

1. While the above example shows that variable bottom plate connections may be specified based on differing shear transfer requirements for portions of the wall, it is acceptable practice to use a constant (i.e., worst-case) base shear connection to simplify construction. However, this can result in excessive fastening requirements for certain loading conditions and shear wall configurations.
2. For the assumed wind loading of 3,000 lb, the wall has excess design capacity (i.e., 4,237 lb). The design wind load may be distributed to the shear wall segments in proportion to their design capacity (as shown in the next step for hold-down design) to reduce the shear connection loads accordingly. For seismic design, this should not be done and the base shear connection design should be based on the design capacity of the shear walls to ensure that a "balanced design" is achieved (i.e., the base connection capacity meets or exceeds that of the shear wall). This approach is necessary in seismic design because the actual shear force realized in the connections may be substantially higher than anticipated by the design seismic load calculated using an R factor in accordance with Equation 3.8-1 of Chapter 3. Refer also to the discussion on R factors and overstrength in Section 3.8.4 of Chapter 3. It should be realized that the GWB interior finish design shear capacity was not included in determining the design shear wall capacity for seismic loading. While this is representative of current building code practice, it can create a situation where the actual shear wall capacity and connection forces experienced are higher than those used for design purposes. This condition (i.e., underestimating of the design shear wall capacity) should also be considered in providing sufficiently strong overturning connections (i.e., hold-downs) as covered in the next step.

3. Determine the chord tension and compression (i.e., overturning) forces in the shear wall segments (Section 6.5.2.4)

Basic equation for overturning (Equation 6.5-7c)

$T = C = (d/x)(F'_s)(h)$

Segment 1

h = 8 ft
d = 3 ft
x = d – (width of end studs + offset to center of hold-down anchor bolt)*
= 3 ft – (4.5 in + 1.5 in)(1ft/12 in) = 2.5 ft

*If an anchor strap is used, the offset dimension may be reduced from that determined above assuming a side-mounted hold-down bracket. Also, depending on the number of studs at the end of the wall segment and the type of bracket used, the offset dimension will vary and must be verified by the designer.

$F'_{s,1,wind}$ = 307 plf
$F'_{s,1,seismic}$ = 218 plf

T = C = (3 ft / 2.5 ft)(307 plf)(8 ft) = 2,947 lb (wind)
T = C = (3 ft / 2.5 ft)(218 plf)(8 ft) = 2,093 lb (seismic)

Segment 2

h = 8 ft
d = 2 ft
x = 2 ft – 0.5 ft = 1.5 ft
$F'_{s,2,wind}$ = 250 plf
$F'_{s,2,seismic}$ = 178 plf

T = C = (2 ft / 1.5 ft)(250 plf)(8 ft) = 2,667 lb (wind)
T = C = (2 ft / 1.5 ft)(178 plf)(8 ft) = 1,899 lb (seismic)

Segment 3

h = 8 ft
d = 8 ft
x = 8 ft – 0.5 ft = 7.5 ft
$F'_{s,2,wind}$ = 352 plf
$F'_{s,2,seismic}$ = 250 plf

T = C = (8 ft / 7.5 ft)(352 plf)(8 ft) = 3,004 lb (wind)
T = C = (8 ft / 7.5 ft)(250 plf)(8 ft) = 2,133 lb (seismic)

Notes:

1. In each of the above cases, the seismic tension and compression forces on the shear wall chords are less than that determined for the wind loading condition. This occurrence is the result of using a larger safety factor to determine the shear wall design capacity and the practice of not including the interior sheathing (GWB) design shear capacity for seismic design. Thus, the chord forces based on the seismic shear wall design capacity may be under-designed unless a sufficient safety factor is used in the manufacturer's rated hold-down capacity to compensate. In other words, the ultimate capacity of the hold-down connector should be greater than the overturning force that could be created based on the ultimate shear capacity of the wall, including the contribution of the interior GWB finish. This condition should be verified by the designer since the current code practice may not provide explicit guidance on the issue of balanced design on the basis of system capacity (i.e., connector capacity relative to shear wall capacity). This issue is primarily a concern with seismic design because of the higher safety factor used to determine design shear wall capacity and the code practice not to include the contributing shear capacity of the interior finish.
2. The compression chord force should be recognized as not being a point load at the top of the stud(s) comprising the compression chord. Rather, the compression chord force is accumulated through the sheathing and begins at the top of the wall with a value of zero and increases to C (as determined above) at the base of the compression chord. Therefore, this condition will affect how the compression chord is modeled from the standpoint of determining its capacity as a column using the column equations in the NDS.
3. The design of base shear connections and overturning forces assume that the wind uplift forces at the base of the wall are offset by 0.6 times the dead load (ASD) at that point in the load path or that an additional load path for uplift is provided by metal strapping or other means.
4. As mentioned in Step 2 for the design of base shear connections, the wind load on the designated shear wall segments may be distributed according to the design capacity of each segment in proportion to that of the total shear wall line. This method is particularly useful when the design shear capacity of the wall line is substantially higher than the shear demand required by the wind load as is applicable to this hypothetical example. Alternatively, a shear wall segment may be eliminated from the analysis by not specifying restraining devices for the segment (i.e., hold-down brackets). If the former approach is taken, the wind load is distributed as follows:

Fraction of design wind load to Segment 1:

$F_{ssw,1,wind} / F_{ssw,total,wind}$ = (921 lb)/(4,237 lb) = 0.22

Fraction of wind load to Segment 2:

$F_{ssw,2,wind} / F_{ssw,total,wind}$ = (500 lb)/(4,237 lb) = 0.12

Fraction of wind load to Segment 3:

$F_{ssw,3,wind} / F_{ssw,total,wind}$ = (2,816 lb)/(4,237 lb) = 0.66

Thus, the unit shear load on each shear wall segment due to the design wind shear of 3,000 lb on the total wall line is determined as follows:

Segment 1: 0.22(3,000 lb)/(3 ft) = 220 plf
Segment 2: 0.12(3,000 lb)/(2 ft) = 180 plf
Segment 3: 0.66(3,000 lb)/(8 ft) = 248 plf

Now, the overturning forces (chord forces) determined above and the base shear connection requirements determined in Step 2 may be recalculated by substituting the above values, which are based on the design wind loading. This approach only applies to the wind loading condition when the design wind loading on the wall line is less than the design capacity of the wall line. As mentioned, it may be more efficient to eliminate a designed shear wall segment to bring the total design shear capacity more in line with the design wind shear load on the wall. Alternatively, a lower capacity shear wall construction may be specified to better match the loading condition (i.e., use a thinner wood structural sheathing panel, etc.). This decision will depend on the conditions experienced in other walls of the building such that a single wall construction type may be used throughout for all exterior walls (i.e., simplified construction).

4. Determine the load-drift behavior of the wall line.

Only the load-drift behavior for wind design is shown below. For seismic design, a simple substitution of the design shear capacities of the wall segments and the safety factor for seismic design (as determined previously) may be used to determine a load-drift relationship for use in seismic design.

The basic equation for load-drift estimation of a shear wall segment is as follows:

$$\Delta = 2.2\left(\frac{0.5}{G}\right)\sqrt[4]{a}\left(\frac{V_d}{F_{SSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \qquad \text{(Equation 6.5-9)}$$

h = 8 ft
G = 0.42 (Spruce-Pine-Fir)

Aspect ratios for the wall segments

$a_1 = 2.67$
$a_2 = 4.0$
$a_3 = 1.0$

$F_{ssw,ult,1,wind} = F_{ssw,1,wind}(SF) = (921 \text{ lb})(2.0) = 1{,}842 \text{ lb}$
$F_{ssw,ult,2,wind} = F_{ssw,2,wind}(SF) = (500 \text{ lb})(2.0) = 1{,}000 \text{ lb}$
$F_{ssw,ult,3,wind} = F_{ssw,3,wind}(SF) = (2{,}816 \text{ lb})(2.0) = 5{,}632 \text{ lb}$

Therefore, the total ultimate capacity of the wall for wind loading is

$F_{ssw,ult,wall,wind}$ = 1,842 lb + 1,000 lb + 5,632 lb = 8,474 lb

Substituting the above values into the basic load-drift equation above, the following load-drift equations are determined for each segment:

Segment 1: $\Delta_1 = 2.41x10^{-9} (V_{d,1,wind})^{2.8}$ (inches)
Segment 2: $\Delta_2 = 1.45x10^{-8} (V_{d,2,wind})^{2.8}$ (inches)
Segment 1: $\Delta_3 = 2.41x10^{-10} (V_{d,3,wind})^{2.8}$ (inches)

Realizing that each segment must deflect equally (or nearly so) as the wall line deflects, the above deflections may be set equivalent to the total wall line drift as follows:

$\Delta_{wall} = \Delta_1 = \Delta_2 = \Delta_3$

Further, the above equations may be solved for V_d as follows:

Segment 1: $V_{d,1,wind} = 1,196 (\Delta_{wall})^{0.36}$
Segment 2: $V_{d,2,wind} = 630 (\Delta_{wall})^{0.36}$
Segment 3: $V_{d,3,wind} = 1,997 (\Delta_{wall})^{0.36}$

The sum of the above equations must equal the wind shear load (demand) on the wall at any given drift of the wall as follows:

$V_{d,wall,wind} = V_{d,1,wind} + V_{d,2,wind} + V_{d,3,wind} = 3,823 (\Delta_{wall})^{0.36}$

Solving for Δ_{wall}, the following final equation is obtained for the purpose of estimating drift and any given wind shear load from zero to $F_{ssw,ult,wall,wind}$:

$\Delta_{wall} = 9.32x10^{-11}(V_{d,wall,wind})^{2.8}$

For the design wind load on the wall of 3,000 lb as assumed in this example, the wall drift is determined as follows:

$\Delta_{wall} = 9.32x10^{-11}(3,000)^{2.8} = 0.51$ inches

Note: This analysis, as with most other methods of determining drift, may overlook many factors in the as-built construction that serve to increase or decrease drift. As discussed in Section 6.2, whole building tests seem to confirm that drift is generally over-predicted.

Conclusion

In this example, the determination of the design shear capacity of a segmented shear wall was presented for seismic design and wind design applications. Issues related to connection design for base shear transfer and overturning forces (chord tension and compression) were also discussed and calculations were made to estimate these forces using a conventional design approach. In particular, issues related to capacity-based design and "balanced design" of connections were discussed. Finally, a method to determine the load-drift behavior of a segmented shear wall line was presented. The final design may vary based on designer decisions and judgments (as well as local code requirements) related to the considerations and calculations as given in this example.

EXAMPLE 6.2 Perforated Shear Wall Design

Given

The perforated shear wall, as shown in the figure below, is essentially the same wall used in Example 6.1. The following dimensions are used:

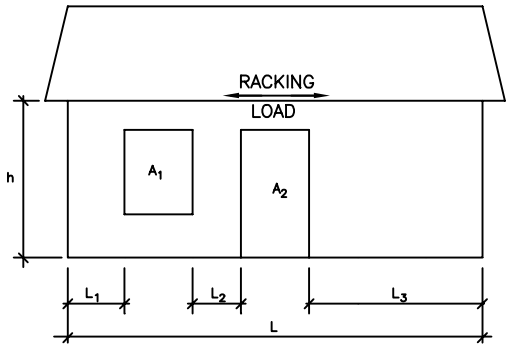
h = 8 ft
L_1 = 3 ft
L_2 = 2 ft
L_3 = 8 ft
L = 19 ft
A_1 = 3.2 ft x 5.2 ft = 16.6 sf (rough window opening area)
A_2 = 3.2 ft x 6.8 ft = 21.8 sf (rough door opening area)

Wall construction:

- Exterior sheathing is 7/16-inch-thick OSB with 8d pneumatic nails (0.113 inch diameter by 2 3/8 inches long) spaced 6 inches on center on panel edges and 12 inches on center in panel field
- Interior sheathing is 1/2-inch-thick gypsum wall board with #6 screws at 12 inches on center
- Framing lumber is Spruce-Pine-Fir, Stud grade (specific gravity, G = 0.42); studs are spaced at 16 inches on center.

Loading condition (assumed for illustration):

Wind shear load on wall line = 3,000 lb
Seismic shear load on wall line = 1,000 lb



Find

1. Design capacity of the perforated shear wall line for wind and seismic shear resistance.
2. Base shear connection requirements.
3. Chord tension and compression forces.
4. Load-drift behavior of the perforated shear wall line and estimated drift at design load conditions.

Solution

1. Determine the factored and adjusted (design) shear capacity for the perforated shear wall line.

$F'_s = F_s\ C_{sp}\ C_{ns}\ [1/SF]$ (Eq. 6.5-1a)

C_{sp} = [1-(0.5-0.42)] = 0.92 (Section 6.5.2.3)
C_{ns} = 0.75 (Table 6.7)
SF = 2.0 (wind design) or 2.5 (seismic design) (Table 6.5)

$F_s = F_{s,ext} + F_{s,int}$ (Section 6.5.2.1)

$F_{s,ext}$ = 905 plf (Table 6.1)
$F_{s,int}$ = 80 plf (Table 6.3)

For wind design

$F_{s,wind}$ = 905 plf + 80 plf = 985 plf
$F'_{s,wind}$ = (985 plf)(0.92)(0.75)(1/2.0) = 340 plf

For seismic design

$F_{s,seismic}$ = 905 plf + 0 plf = 905 plf
$F'_{s,seismic}$ = (905 plf)(0.92)(0.75)(1/2.5) = 250 plf

The design capacity of the perforated shear wall is now determined as follows:

$F_{psw} = F'_s\ C_{op}\ C_{dl}\ L$ (Eq. 6.5-1b)

where,

$C_{op} = r/(3-2r)$
$r = 1/(1+\alpha/\beta)$
$\alpha = \Sigma A_o/(h \times L) = (A_1 + A_2)/(h \times L)$
$= (16.6\ sf + 21.8\ sf)/(8\ ft)(19\ ft) = 0.25$
$\beta = \Sigma L_i/L = (L_1 + L_2 + L_3)/L$
$= (3\ ft + 2\ ft + 8\ ft)/(19\ ft) = 0.68$
$r = 1/(1+0.25/0.68) = 0.73$
$C_{op} = 0.73/(3-2(0.73)) = 0.47$

$C_{dl} = 1 + 0.15(w_D/300) \leq 1.15$

Assume for the sake of this example that the roof dead load supported at the top of the wall is 225 plf and that the design wind uplift force on the top of the wall is 0.6(225 plf) – 400 plf = -265 plf (net design uplift). Thus, for wind design in this case, no dead load can be considered on the wall and the C_{dl} factor does not apply for calculation of the perforated shear wall resistance to wind loads. It does apply to seismic design, as follows:

$w_D = 0.6*(225\ plf) = 135\ plf$

*The 0.6 factor comes from the load combinations 0.6D + (W or 0.7E) or 0.6D – W_u as given in Chapter 3.

$C_{dl} = 1 + 0.15(135/300) = 1.07$

For wind design,

$F_{psw,wind} = (340\ plf)(0.47)(1.0)(19\ ft) = 3{,}036\ lb$

For seismic design,

$F_{psw,seismic} = (250\ plf)(0.47)(1.07)(19\ ft) = 2{,}389\ lb$

Note: In Example 6.1 using the segmented shear wall approach, the design shear capacity of the wall line was estimated as 4,237 lb (wind) and 3,010 lb (seismic) when all of the segments were restrained against overturning by use of hold-down devices. However, given that the design shear load on the wall is 3,000 lb (wind) and 1,000 lb (seismic), the perforated shear wall design capacity as determined above is adequate, although somewhat less than that of the segmented shear wall. Therefore, hold-downs are only required at the wall ends (see Step 3).

2. Determine the base shear connection requirement for the perforated shear wall.

If the wall had a continuous bottom plate that serves as a distributor of the shear forces resisted by various portions of the wall, the base shear connection could be based on the perforated shear wall's design capacity as determined in Step 1 as follows:

For wind design,

Uniform base shear = (3,036 lb)/19 ft = 160 plf

For seismic design,

Uniform base shear = (2,389 lb)/19 ft = 126 plf

However, the wall bottom plate is not continuous in this example and, therefore, the base shears experienced by the portions of the wall to the left and right of the door opening are different as was the case in the segmented shear wall design approach of Example 6.1. As a conservative solution, the base shear connection could be designed to resist the design unit shear capacity of the wall construction, $F'_{s,wind} = 340\ plf$ or $F'_{s,seismic} = 250\ plf$. Newer codes that recognize the perforated shear method may require this more conservative approach to be used when the bottom plate is not continuous such that it serves as a distributor (i.e., similar in function to a shear wall collector except shear transfer is out of the wall instead of into the wall). Of course, the bottom plate must be continuous and any splices must be adequately detailed in a fashion similar to collectors (see Example 6.3).

As an alternative, the portion of the wall to the left of the door opening can be treated as a separate perforated shear wall for the left-to-right loading condition. In doing so, the design shear capacity of the left portion of the wall may be determined to be 1,224 lb and the base shear connection required is (1,224 lb)/8ft = 153 plf, much less than the 340 lb required in the wind load condition. The right side of the wall is solid sheathed and, for the right-to-left loading condition, the base shear is equivalent to the design shear capacity of the wall or 340 plf. These calculations can also be performed using the seismic design values for the perforated shear wall. This approach is based on the behavior of a perforated shear wall where the leading edge and the immediately adjacent shear wall segments are fully restrained as in the segmented shear wall approach for one direction of loading. Thus, these segments will realize their full unit shear capacity for one direction of loading. Any interior segments will contribute, but at a reduced amount do to the reduced restraint condition. This behavior is represented in the adjustment provided by the C_{op} factor which is the basis of the perforated shear wall method. Unfortunately, the exact distribution of the uplift forces and shear forces within the wall are not known. It is for this reason that they are assigned conservative values for design purposes. Also, to accommodate potential uplift forces on the bottom plate in the regions of interior perforated shear wall segments, the base shear connections are required to resist an uplift load equivalent to the design unit shear capacity of the wall construction. In the case of this example, the base shear connection would need to resist a shear load of 340 plf (for the wind design condition) and an uplift force of 340 plf (even if under a zero wind uplift load).

Testing has shown that for walls constructed similar to the one illustrated in this example, a bottom plate connection of 2 16d pneumatic nails (0.131 inch diameter by 3 inches long) at 16 inches on center or 5/8-inch-diameter anchor bolts at 6 feet on center provides suitable shear and uplift resistance – at least equivalent to the capacity of the shear wall construction under conditions of no dead load or wind uplift (NAHBRC, 1999). For other conditions, this connection must be designed following the procedures given in Chapter 7 using the conservative assumptions as stated above.

As an alternative base connection that eliminates the need for hold-down brackets at the ends of the perforated shear wall, straps can be fastened to the individual studs to resist the required uplift force of 340 plf as applicable to this example. If the studs are spaced 16 inches on center, the design capacity of the strap must be (340 plf)(1.33 ft/stud) = 452 lb per stud. If an uplift load due to wind uplift on the roof must also be transferred through these straps, the strap design capacity must be increased accordingly. In this example, the net wind uplift at the top of the wall was assumed to be 265 plf. At the base of the wall, the uplift is 265 plf – 0.6(8 ft)(8 psf) = 227 plf. Thus, the total design uplift restraint must provide 340 plf + 227 plf = 567 plf. On a per stud basis (16 inch on center framing), the design load is 1.33 ft/stud x 567 plf = 754 lb/stud. This value must be increased for studs adjacent to wall openings where the wind uplift force in increased. This can be achieved by using multiple straps or by specifying a larger strap in these locations. Of course, the above combination of uplift loads assumes that the design wind uplift load on the roof occurs simultaneously with the design shear load on the wall. However, this condition is not usually representative of actual conditions depending on wind orientation, building configuration, and the shear wall location relative to the uplift load paths.

3. Determine the chord tension and compression forces

The chord tension and compression forces are determined following the same method as used in Example 6.1 for the segmented shear wall design method, but only for the first wall segment in the perforated shear wall line (i.e. the restrained segment). Therefore, the tension forces at the end of the wall are identical to those calculated in Example 6.1 as shown below:

Left end of the wall (Segment 1 in Example 6.1):

T = 2,947 lb (wind design)
T = 2,093 lb (seismic design)

Right end of the wall (Segment 3 in Example 6.1):

T = 3,004 lb (wind design)
T = 2,133 lb (seismic design)

Note: One tension bracket (hold-down) is required at each the end of the perforated shear wall line and not on the interior segments. Also, refer to the notes in Example 6.1 regarding "balanced design" of overturning connections and base shear connections, particularly when designing for seismic loads.

4. Determine the load-drift behavior of the perforated shear wall line.

The basic equation for load-drift estimation of a perforated shear wall line is as follows (Section 6.5.2.6):

$$\Delta = 1.8\left(\frac{0.5}{G}\right)\left(\frac{1}{\sqrt{r}}\right)\left(\frac{V_d}{F_{PSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \qquad \text{(Eq. 6.5-8)}$$

h = 8 ft
G = 0.42(specific gravity for Spruce-Pine-Fir)
r = 0.73 (sheathing area ratio determined in Step 1)

$F_{psw,ult,wind}$ = $(F_{psw,wind})(SF)$ = (3,036 lb)(2.0) = 6,072 lb
$F_{psw,ult,seismic}$ = $(F_{psw,seismic})(SF)$ = (2,389 lb)(2.5) = 5,973 lb

Substituting in the above equation,

$\Delta_{wind} = 6.4x10^{-11}(V_{d,wind})^{2.8}$
$\Delta_{seismic} = 6.7x10^{-11}(V_{d,seismic})^{2.8}$

For the design wind load of 3,000 lb and the design seismic load of 1,000 lb (assumed for the purpose of this example), the drift is estimated as follows:

$\Delta_{wind} = 6.4x10^{-11}(3{,}000)^{2.8} = 0.35$ inches
$\Delta_{seismic} = 6.7x10^{-11}(1{,}000)^{2.8} = 0.02$ inches

Note: The reader is reminded of the uncertainties in determining drift as discussed in Example 6.1 and also in Chapter 6. For seismic design, some codes may require the design seismic drift to be amplified (multiplied by) a factor of 4 to account for the potential actual forces that may be experienced relative to the design forces that are determined using an R factor; refer to Chapter 3 for additional discussion. Thus, the amplified drift may be determined as 4 x 0.02 inches = 0.08 inches. However, if the seismic shear load is magnified (i.e., 4 x 1,000 lb = 4,000 lb) to account for a possible actual seismic load (not modified for the seismic response of the shear wall system), the seismic drift calculated in the above equation becomes 0.8 inches which is an order of magnitude greater. The load adjustment is equivalent to the use of an R of 1.5 instead of 6 in Equation 3.8-1 of Chapter 3. However, this latter approach of magnifying the load

is not currently required in the existing building codes for drift determination. As mentioned, drift is not usually considered in residential design. Finally, the above equations may be used to determine a load-drift curve for a perforated shear wall for values of V_d ranging from 0 to $F_{psw,ult}$. While the curve represents the non-linear behavior of a perforated shear wall, it should only be considered as a representation, and not an exact solution.

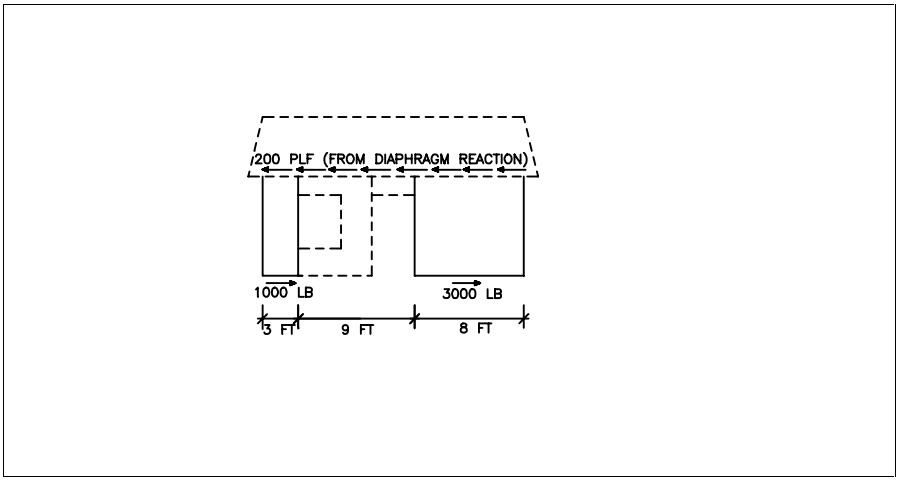
Conclusion

In this example, the determination of the design shear capacity of a perforated shear wall was presented for seismic design and wind design applications. Issues related to connection design for base shear transfer and overturning forces (chord tension) were also discussed and calculations (or conservative assumptions) were made to estimate these forces. In particular, issues related to capacity-based design and "balanced design" of connections were discussed. Finally, a method to determine the load-drift behavior of a perforated shear wall line was presented. The final design may vary based on designer decisions and judgments (as well as local code requirements) related to the considerations and calculations as given in this example.

EXAMPLE 6.3 Shear Wall Collector Design

Given

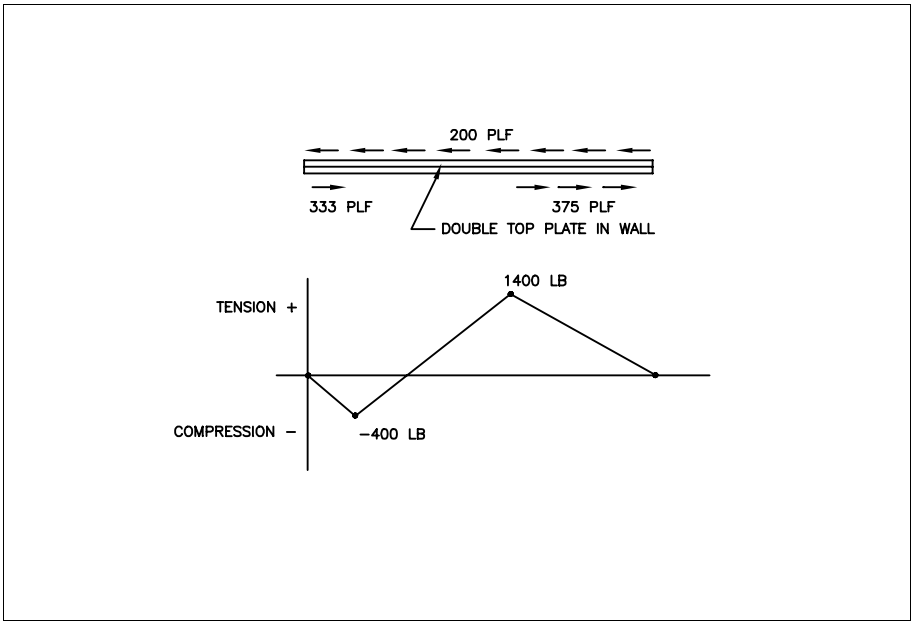
The example shear wall, assumed loading conditions, and dimensions are shown in the figure below.



Find The maximum collector tension force

Solution

1. The collector force diagram is shown below based on the shear wall and loading conditions in the figure above.



The first point at the interior end of the left shear wall segment is determined as follows:

200 plf (3 ft) – 333 plf (3 ft) = - 400 lb (compression force)

The second point at the interior end of the right shear wall segment is determined as follows:

- 400 lb + 200 plf (9 ft) = 1,400 lb (tension force)

The collector load at the right-most end of the wall returns to zero as follows:

1,400 lb – 375 plf (8 ft) + 200 plf (8 ft) = 0 lb

Conclusion

The maximum theoretical collector tension force is 1,400 lb at the interior edge of the 8-foot shear wall segment. The analysis does not consider the contribution of the "unrestrained" wall portions that are not designated shear wall segments and that would serve to reduce the amount of tension (or compression) force developed in the collector. In addition, the load path assumed in the collector does not consider the system of connections and components that may share load with the collector (i.e., wall sheathing and connections, floor or roof construction above and their connections, etc.). Therefore, the collector load determined by assuming the top plate acts as an independent element can be considered very conservative depending on the wall-floor/roof construction conditions. Regardless, it is typical practice to design the collector (and any splices in the collector) to resist a tension force as calculated in this example. The maximum compressive force in the example collector is determined by reversing the loading direction and is equal in magnitude to the maximum tension force. Compressive forces are rarely a concern when at least a double top plate is used as a collector, particularly when the collector is braced against lateral buckling by attachment to other construction (as would be generally necessary to deliver the load to the collector from somewhere else in the building).

EXAMPLE 6.4 Horizontal (Floor) Diaphragm Design

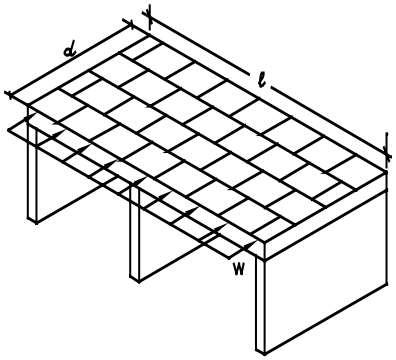
Given

The example floor diaphragm and its loading and support conditions are shown in the figure below. The relevant dimensions and loads are as follows:

d	= 24 ft	
l	= 48 ft	
w	= 200 plf	(from wind or seismic lateral load)*

*Related to the diaphragm's tributary load area; see Chapter 3 and discussions in Chapter 6.

The shear walls are equally spaced and it is assumed that the diaphragm is flexible (i.e. experiences beam action) and that the shear wall supports are rigid. This assumption is not correct because the diaphragm may act as a "deep beam" and distribute loads to the shear wall by "arching" action rather than bending action. Also, the shear walls cannot be considered to be perfectly rigid or to exhibit equivalent stiffness except when designed exactly the same with the same interconnection stiffness and base support stiffness. Regardless, the assumptions made in this example are representative of typical practice.



Find

1. The maximum design unit shear force in the diaphragm (assuming simple beam action) and the required diaphragm construction.
2. The maximum design moment in the diaphragm (assuming simple beam action) and the associated chord forces.

Solution

1. The maximum shear force in the diaphragm occurs at the center shear wall support. Using the beam equations in Appendix A for a 2-span beam, the maximum shear force is determined as follows:

$$V_{max} = \frac{5}{8} w\left(\frac{l}{2}\right) = \frac{5}{8}(200\ \text{plf})\left(\frac{48\ \text{ft}}{2}\right) = 3{,}000\ \text{lb}$$

The maximum design unit shear in the diaphragm is determined as follows:

$$v_{max} = \frac{V_{max}}{d} = \frac{3{,}000\ \text{lb}}{24\ \text{ft}} = 125\ \text{plf}$$

From Table 6.8, the lightest unblocked diaphragm provides adequate resistance. Unblocked means that the panel edges perpendicular to the framing (i.e., joists or rafters) are not attached to blocking. The perimeter, however, is attached to a continuous member to resist chord forces. For typical residential floor construction a 3/4-inch-thick subfloor may be used which would provide at least 240 plf of design shear capacity. In typical roof construction, a minimum 7/16-inch-thick sheathing is used which would provide about 230 plf of design shear capacity. However, residential roof construction does not usually provide the edge conditions (i.e., continuous band joist of 2x lumber) associated with the diaphragm values in Table 6.8. Regardless, roof diaphragm performance has rarely (if ever) been a problem in light-frame residential construction and these values are often used to approximate roof diaphragm design values.

Note: The shear forces at other regions of the diaphragm and at the locations of the end shear wall supports can be determined in a similar manner using the beam equations in Appendix A. These shear forces are equivalent to the connection forces that must transfer shear between the diaphragm and the shear walls at the ends of the diaphragm. However, for the center shear wall, the reaction (connection) force is twice the unit shear force in the diaphragm at that location (see beam equations in Appendix A). Therefore, the connection between the center shear wall and the diaphragm in this example must resist a design shear load of 2 x 125 plf = 250 plf. However, this load is very dependent on the assumption of a “flexible” diaphragm and “rigid” shear walls.

2. The maximum moment in the diaphragm also occurs at the center shear wall support. Using the beam equations in Appendix A, it is determined as follows:

$$M_{max} = \frac{1}{8}w\left(\frac{l}{2}\right)^2 = \frac{1}{8}(200\text{plf})\left(\frac{48\text{ft}}{2}\right)^2 = 14{,}400\text{ft} - \text{lb}$$

The maximum chord tension and compression forces are at the same location and are determined as follows based on the principle of a force couple that is equivalent to the moment:

$$T = C = \frac{M_{max}}{d} = \frac{14{,}400\text{ft} - \text{lb}}{24\text{ft}} = 600\text{lb}$$

Therefore, the chord members (i.e., band joist and associated wall or foundation framing that is attached to the chord) and splices must be able to resist 600 lb of tension or compression force. Generally, these forces are adequately resisted by the framing systems bounding the diaphragm. However, the adequacy of the chords should be verified by the designer based on experience and analysis as above.

Conclusion

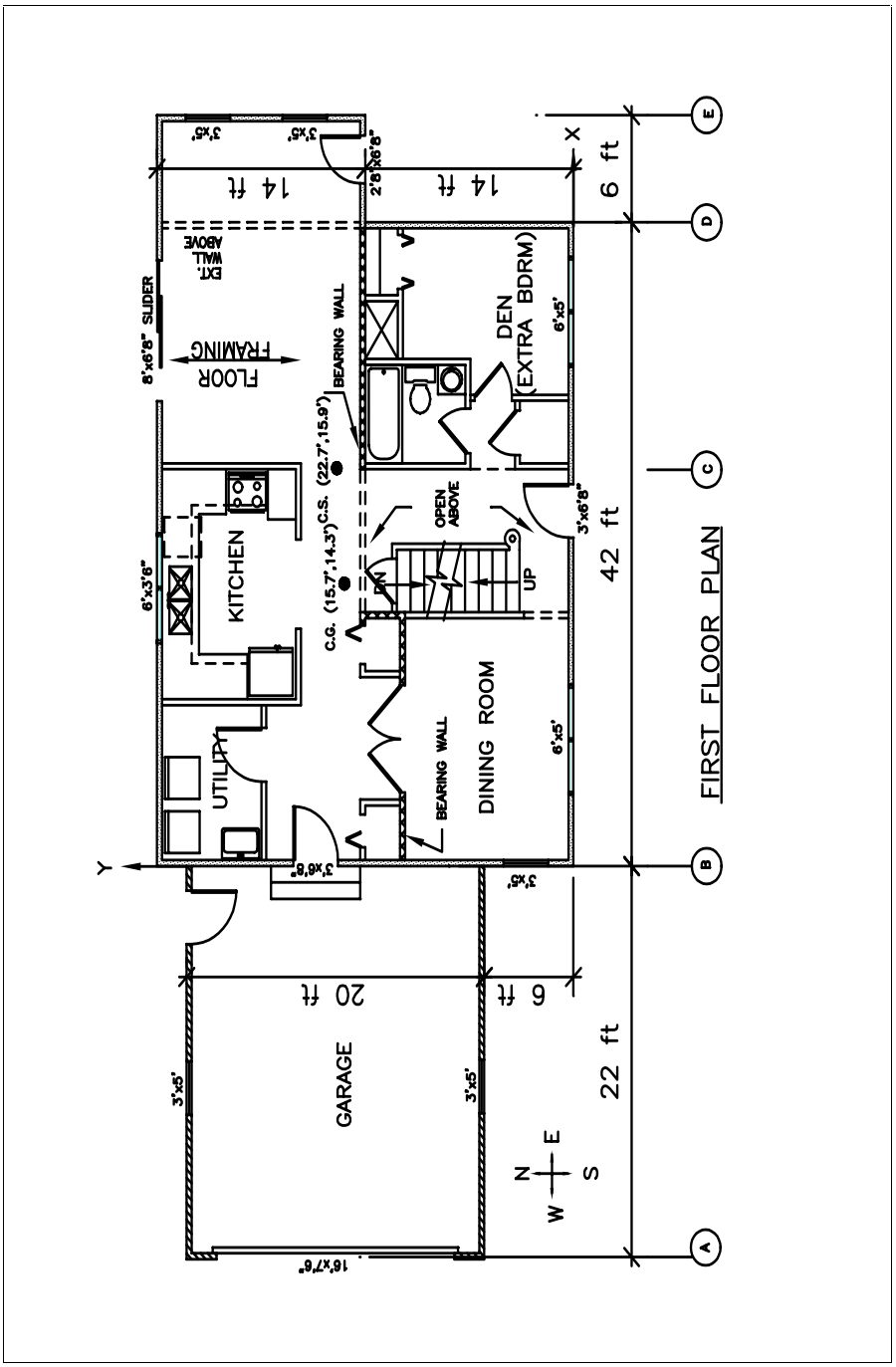
In this example, the basic procedure and principles for horizontal diaphragm design were presented. Assumptions required to conduct a diaphragm analysis based on conventional beam theory were also discussed.

EXAMPLE 6.5 Horizontal Shear Load Distribution Methods

Given

General

In this example, the first floor plan of a typical two-story house with an attached garage (see Figure below) is used to demonstrate the three methods of distributing shear loads discussed in Chapter 6, Section 6.4.2. The first story height is 8 ft (i.e., 8 ft ceiling height). Only the load in the North-South (N-S) direction is considered in the example. In a complete design, the load in the East-West (E-W) direction would also need to be considered.



Lateral Load Conditions

The following design N-S lateral loads are determined for the story under consideration using the methods described in Chapter 3 for wind and seismic loads. A fairly high wind load and seismic load condition is assumed for the purpose of the example.

Design N-S Wind Lateral Load (120 mph gust, exposure B)

House:	17,411	lb total story shear
Garage:	3,928	lb total story shear
Total:	21,339	lb

Design N-S Seismic Lateral Load (mapped S_s = 1.5g)

House:	7,493	lb total story shear (tributary weight is 37,464 lb)
Garage:	1,490	lb total story shear (tributary weight is 7,452 lb)
Total:	8,983	lb

Designation of Shear Walls in N-S Direction

Initially, there are four N-S lines designated in the first story for shear wall construction. The wall lines are A, B, D, and E. If needed, an interior wall line may also be designated and designed as a shear wall (see wall line C in the figure above).

The available length of full-height wall segments in each N-S shear wall line is estimated as follows from the floor plan:

Wall Line A:	2 ft + 2 ft	= 4 ft	(garage return walls)
Wall Line B:	~~1.33 ft~~* + 11 ft + 9 ft	= 20 ft	(garage/house shared wall)
Wall Line D:	14 ft	= 14 ft	(den exterior wall)
Wall Line E:	2 ft + 3 ft + 2 ft	= 7 ft	(living room exterior wall)
Total:		= 45 ft	

*The narrow 1.33 ft segment is not included in the analysis due to the segment's aspect ratio of 8 ft/1.33 ft = 6, which is greater than the maximum allowable of 4. Some current building codes may restrict the segment aspect ratio to a maximum of 2 or 3.5 depending on the code and the edition in local use. In such a case, many of the useable shear wall segments would be eliminated (i.e., all of the 2 ft segments). Thus, the garage opening wall would require larger segments, a portal frame (see Section 6.5.2.7), or transfer of the garage shear load to the house by torsion (i.e., treat the garage as a cantilever projecting from the house under a uniform lateral load).

Find

1. Using the "total shear method" of horizontal shear load distribution, determine the total length of shear wall required and the required shear wall construction in the N-S direction.
2. Using the "tributary area method" of horizontal shear load distribution, determine the shear resistance and wall construction required in each N-S shear wall line.
3. Using the "relative stiffness method" of horizontal shear load distribution, determine the shear loads on the N-S shear wall lines.

Solution

1. Using the total shear approach, determine the unit shear capacity required based on the given amount of available shear wall segments in each N-S wall line and the total N-S shear load.

In this part of the example, it is assumed that the wall lines will be designed as segmented shear wall lines. From the given information, the total length of N-S shear wall available is 45 ft. It is typical practice in this method to not include segments with aspect ratios greater than 2 since stiffness effects on the narrow segments are not explicitly considered. This would eliminate the 2 ft segments and the total available length of shear wall would be 45 ft – 8 ft = 37 ft in the N-S direction.

The required design unit shear capacity of the shear wall construction and ultimate capacity is determined as follows for the N-S lateral design loads:

Wind N-S

$F'_{s,wind}$ = (21,339 lb)/37 ft = 576 plf
$F_{s,\ wind}$ = $(F'_{s,wind})$(SF) = (576 plf)(2.0) = 1,152 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the shear walls must meet or exceed 1,152 plf. Assuming that standard 1/2-thick GWB finish is used on the interior wall surfaces (80 plf minimum from Table 6.3), the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,wind} = F_{s,ext} + F_{s,int}$
$F_{s,ext}$ = 1,152 plf – 80 plf = 1,072 plf

From Table 6.1, any of the wall constructions that use a 4 inch nail spacing at the panel perimeter exceed this requirement. By specifying and 3/8-thick Structural I wood structural panel with 8d common nails spaced at 4 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors. For example, assume the following framing lumber and nails are used in the shear wall construction:

lumber species:	Spruce-Pine-Fir (G=0.42)	C_{sp} = 0.92
nail type:	8d pneumatic, 0.113-inch-diameter	C_{ns} = 0.75

Thus, values in Table 6.1 would need to be multiplied by (0.92)(0.75) = 0.69. This adjustment requires a 15/32-inch-thick sheathing with the 8d nails (i.e., 1,539 plf x 0.69 = 1,062 plf which is close enough to the required 1,072 plf for practical design purposes). Alternatively, a 7/16-inch thick wood structural panel sheathing could be used in accordance with footnote 5 of Table 6.1; however, the horizontal joint between panels would need to be blocked. In extreme lateral load conditions, it may be necessary (and more efficient) to consider a "double sheathed" wall construction (i.e., structural wood panels on both sides of the wall framing) or to consider the addition of an interior shear wall line (i.e., design the interior walls along wall line C as shear walls).

Seismic N-S

$F'_{s,seismic}$ = (8,983 lb)/37 ft = 243 plf
$F_{s,\ seismic}$ = $(F'_{s,seismic})$(SF) = (243 plf)(2.5) = 608 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the wall line must meet or exceed 608 plf. Since seismic codes do not permit the consideration of a 1/2-thick GWB interior finish, the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,seismic} = F_{s,ext}$ = 608 plf

From Table 6.1, any of the wood structural panel wall constructions that use a 6 inch nail spacing at the panel perimeter exceed this requirement. By specifying 3/8-inch-thick Structural I wood structural panels with 8d common nails spaced at 6 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors as demonstrated above for the N-S wind load case.

The base shear connections may be designed in this method by considering the total length of continuous bottom plate in the N-S shear wall lines. As estimated from the plan, this length is approximately 56 feet. Thus, the base connection design shear load (parallel to the grain of the bottom plate) is determined as follows:

Base wind design shear load = (21,339 lb)/(56 ft) = 381 plf
Base seismic design shear load = (8,983 lb)/(56 ft) = 160 plf

The base shear connections may be designed and specified following the methods discussed in Chapter 7 – Connections. A typical 5/8-inch-diameter anchor bolt spaced at 6 feet on center or standard bottom plate nailing may be able to resist as much as 800 plf (ultimate shear capacity) which would provided a "balanced" design capacity of 400 plf or 320 plf for wind and seismic design with safety factors of 2.0 and 2.5, respectively. Thus, a conventional wall bottom plate connection may be adequate for the above condition; refer to Chapter 7 for connection design information and the discussion in Section 7.3.6 for more details on tested bottom plate connections.

If the roof uplift load is not completely offset by 0.6 times the dead load at the base of the first story wall, then strapping to transfer the net uplift from the base of the wall to the foundation or construction below must be provided.

The hold-down connections for the each shear wall segment in the designated shear wall lines are designed in the manner shown in Example 6.1. Any overturning forces originating from shear walls on the second story must also be included as described in Section 6.4.2.4.

Notes:

1. The contribution of the interior walls to the lateral resistance is neglected in the above analysis for wind and seismic loading. As discussed in Chapter 6, these walls can contribute significantly to the lateral resistance of a home and serve to reduce the designated shear wall loads and connection loads through alternate, "non-designed" load paths. In this example, there is approximately 40 ft of interior partition walls in the N-S direction that each have a minimum length of about 8 ft or more (small segments not included). Assuming a design unit shear value of 80 plf / 2 = 40 plf (safety factor of 2), the design lateral resistance may be at least 40 ft x 40 plf = 1,600 lb. While this is not a large amount, it should factor into the design consideration, particularly when a lateral design solution is considered to be marginal based on an analysis that does not consider interior partition walls.
2. Given the lower wind shear load in the E-W direction, the identical seismic story shear load in the E-W direction, and the greater available length of shear wall in the E-W direction, an adequate amount of lateral resistance should be no problem for shear walls in the E-W direction. It is probable that some of the available E-W shear wall segments may not even be required to be designed and detailed as shear wall segments. Also, with hold-down brackets at the ends of the N-S walls that are detailed to anchor a common corner stud (to which the corner sheathing panels on each wall are fastened with the required panel edge fastening), the E-W walls are essentially perforated shear wall lines and may be treated as such in evaluating the design shear capacity of the E-W wall lines.
3. The distribution of the house shear wall elements appears to be reasonably "even" in this example. However, the garage opening wall could be considered a problem if sufficient connection of the garage to the house is not provided to prevent the garage from rotating separately from the house under the N-S wind or seismic load. Thus, the garage walls and garage roof diaphragm should be adequately attached to the house so that the garage and house act as a structural unit. This process will be detailed in the next part of this example.

2. Determine the design shear load on each wall line based on the tributary area method.

Following the tributary area method of horizontal force distribution, the loads on the garage and the house are treated separately. The garage lateral load is assumed to act through the center of the garage and the house load is assumed to act through the center of the house. The extension of the living room on the right side of the plan is only one story and is considered negligible in its impact to the location of the real force center; although, this may be considered differently by the designer. Therefore, the lateral force (load) center on the garage is considered to act in the N-S direction at a location one-half the distance between wall lines A and B (see the given floor plan diagram). Similarly, the N-S force center on the house may be considered to act half-way between wall lines B and D (or perhaps a foot or less farther to the right to compensate for the living room "bump-out"). Now, the N-S lateral design loads are assigned to wall lines A, B, and D/E as follows:

<u>Wall Line A</u>

Wind design shear load = 1/2 garage shear load = 0.5(3,928 lb) = 1,964 lb
Seismic design shear load = 0.5(1,490 lb) = 745 lb

<u>Wall Line B</u>

Wind design shear load = 1/2 garage shear load + 1/2 house shear load
= 1,964 lb + 0.5(17,411 lb) = 10,670 lb
Seismic design shear load = 745 lb + 0.5(7,493 lb) = 4,492 lb

Wall Line D/E

Wind design shear load = 1/2 house shear load = 0.5(17,411 lb) = 8,706 lb
Seismic design shear load = 0.5(7,493 lb) = 3,747 lb

Based on the design shear loads above, each of the wall lines may be designed in a fashion similar to that used in Step 1 (total shear method) by selecting the appropriate wall construction to meet the loading demand. For example, the design of wall line B would proceed as shown below (using the perforated shear wall method in this case) for the required wind shear load.

The following equations are used to determine the required ultimate shear capacity, F_s, of the wall construction (interior and exterior sheathing type and fastening):

$F'_s = [(F_{s,ext})(C_{sp})(C_{ns}) + F_{s,int}]x[1/SF]$ (based on Eq. 6.5-1a)
$F_{psw} = F'_s\, C_{op}\, C_{dl}\, [L]$ (Eq. 6.5-1b)

Substituting the first equation above into the second,

$F_{psw} = [(F_{s,ext})(C_{sp})(C_{ns}) + F_{s,int}]\, [1/SF]\, C_{op}\, C_{dl}\, [L]$

To satisfy the design wind shear load requirement for Wall Line B,

$F_{psw} \geq 10{,}670$ lb

Assume that the wall construction is the same as used in Example 6.2. The following parameters are determined for Wall Line B:

$C_{sp} = 0.92$	(Spruce-Pine-Fir)
$C_{ns} = 0.75$	(8d pneumatic nail, 0.113-inch-diameter)
$C_{dl} = 1.0$	(zero dead load due to wind uplift)
$SF = 2.0$	(wind design safety factor)
$C_{op} = 0.71$	(without the corner window and narrow segment)*
$L = 28$ ft – 1.33 ft – 3 ft = 23.67 ft	(length of perforated shear wall line)*
$F_{s,int} = 80$ plf	(Table 6.3, minimum ultimate unit shear capacity)

*The perforated shear wall line begins at the interior edge of the 3’ x 5’ window opening because the wall segment adjacent to the corner exceeds the maximum aspect ratio requirement of 4. Therefore, the perforated shear wall is “embedded” in the wall line.

Substituting the values above into the equation for F_{psw} , the following value is obtained for $F_{s,ext}$:

$10{,}670 \text{ lb} = [(F_{s,ext})(0.92)(0.75) + 80 \text{ plf}]\, [1/2.0]\, (0.71)\, (1.0)\, [23.67 \text{ ft}]$

$F_{s,ext} = 1{,}724$ plf

By inspection in Table 6.1, the above value is achieved for a shear wall constructed with 15/32-inch-thick Structural 1 wood structural panel sheathing with nails spaced at 3 inches on the panel edges. The value is 1,722 plf which is close enough for practical purposes (particularly given that contribution of interior walls is neglected in the above analysis). Also, a thinner sheathing may be used in accordance with Footnote 5 of Table 6.1. As another alternative, wall line B could be designed as a segmented shear wall. There are two large shear wall segments that may be used. In total they are 20 ft long. Thus, the required ultimate shear capacity for wall line B using the segmented shear wall method is determined as follows:

$F'_s = (F_{s,ext}\, C_{sp}\, C_{ns} + F_{s,int})\, C_{ar}$ [1/SF] (based on Eq. 6.5-2a)
$F_{ssw} = F'_s \times L$ (Eq. 6.5-2b)
$F_{ssw} \geq 10{,}670$ lb (wind load requirement on wall line B)

Substituting the first equation into the second

$$F_{ssw} = (F_{s,ext}\, C_{sp}\, C_{ns} + F_{s,int})\, C_{ar}\, [1/SF] \times L$$

The following parameter values are used:

$C_{sp} = 0.92$ (same as before)
$C_{ns} = 0.75$ (same as before)
$C_{ar} = 1.0$ (both segments have aspect ratios less than 2)*
SF = 2.0 (for wind design)
L = 20 ft (total length of the two shear wall segments)*
$F_{s,int} = 80$ plf (minimum ultimate unit shear capacity)

*If the wall segments each had different values for C_{ar} because of varying adjustments for aspect ratio, then the segments must be treated independently in the equation above and the total length could not be summed as above to determine a total L.

Now, solving the above equations for $F_{s,ext}$ the following is obtained:

$$10{,}670 \text{ lb} = [(F_{s,ext})(0.92)(0.75) + 80 \text{ plf}](1.0)[1/2.0](20 \text{ ft})$$

$$F_{s,ext} = 1{,}430 \text{ plf}$$

By inspection of Table 6.1 using the above value of $F_{s,ext}$, a 4 inch nail spacing may be used to meet the required shear loading in lieu of the 3 inch nail spacing used if the wall were designed as a perforated shear wall. However, two additional hold down brackets would be required in Wall Line B to restrain the two wall segments as required by the segmented shear wall design method.

Wall Line A poses a special design problem since there are only two narrow shear wall segments to resist the wind design lateral load (1,964 lb). Considering the approach above for the segmented shear wall design of Wall Line B and realizing that $C_{ar} = 0.71$ (aspect ratio of 4), the following value for $F_{s,ext}$ is obtained for Wall Line A:

$$F_{ssw} = (F_{s,ext}\, C_{sp}\, C_{ns} + F_{s,int})\, C_{ar}\, [1/SF] \times L$$

$$1{,}964 \text{ lb} = [(F_{s,ext})(0.92)(0.75) + 0^*](0.71)[1/2.0](4 \text{ ft})$$

*The garage exterior walls are assumed not to have interior finish. The shared wall between the garage and the house, however, is required to have a fire rated wall which is usually satisfied by the use of 5/8-thick gypsum wall board. This fire resistant finish is placed over the wood structural sheathing in this case and the impact on wall thickness (i.e. door jamb width) should be considered by the architect and builder.

Solving for $F_{s,ext}$,

$$F_{s,ext} = 2{,}004 \text{ plf}$$

By inspecting Table 6.1, this would require 15/32-inch-thick wood structural panel with nails spaced at 2 inches on center and would require 3x framing lumber (refer to footnote 3 of Table 6.1). However, the value of C_{ns} (=0.75) from Table 6.7 was based on a 0.113-inch diameter nail for which the table does not give a conversion relative to the 10d common nail required in Table 6.1. Therefore, a larger nail should be used at the garage opening. Specifying an 8d common nail or similar pneumatic nail with a diameter of 0.131 inches (see Table 6.7), a C_{ns} value of 1.0 is used and $F_{s,ext}$ may be recalculated as above to obtain the following:

$$F_{ssw} = (F_{s,ext}\, C_{sp}\, C_{ns} + F_{s,int})\, C_{ar}\, [1/SF] \times L$$

$$1{,}964 \text{ lb} = [(F_{s,ext})(0.92)(1.0) + 0](0.71)[1/2.0](4 \text{ ft})$$

$$F_{s,ext} = 1{,}503 \text{ plf}$$

Inspecting Table 6.1 again, it is now found that 15/32-inch-thick wood structural panel sheathing with 8d common nails spaced at 4 inches on center provides an ultimate rated unit shear capacity of 1,539 plf > 1,503 plf. This design does not require the use of 3x framing lumber which allows the same lumber to be used for all wall construction. The only added detail is the difference in nail type and spacing for the garage return walls. From the standpoint of simplicity, the easiest solution would be to increase the width of the garage shear wall segments; however, design simplicity is not always the governing factor. Also, a portal frame system may be designed based on the information and references provided in Section 6.5.2.7.

Finally, the garage should be adequately tied to the building to ensure that the garage section and the house section act as a structural unit. This may be achieved by fastening the end rafter or truss top chord in the roof to the house framing using fasteners with sufficient withdrawal capacity (i.e. ring shank nails or lag screws). The same should be done for the end studs that are adjacent to the house framing. Ideally, the garage roof diaphragm may be tied into the house second floor diaphragm by use of metal straps and blocking extending into the floor diaphragm and garage roof diaphragm a sufficient distance in each direction (i.e., 4 feet). With sufficient connection to the house end wall and floor diaphragm, the garage opening issue may be avoided completely. The connection load to the house discussed above can then be determined by treating the garage roof diaphragm as a cantilevered horizontal beam on the side of the home with a fixed end moment at the connection to the house. The fixed end moment (assuming the garage opening provides no lateral shear resistance) is determined based on the beam equation for a cantilever beam (see Appendix A). For the wind load on the garage, the fixed end moment due to lateral load is (3,928 lb)(11 ft) = 43,208 ft-lb. This moment may be resisted by a strap at either side of the garage roof with about a 2,500 lb design tension capacity (i.e. 43,208 ft-lb/18 ft = 2,400 lb). Preferably, the strap would be anchored to the garage roof diaphragm and house floor diaphragm as described above. Alternatively, this moment could be resisted by numerous lag screws or similar fasteners attaching the garage framing to the house framing. By this method, the garage end walls would require no special shear wall design. Of course, connections required to resist wind uplift and transverse shear loads on the garage door and return walls would still be required.

For the seismic design lateral loads in this example, the garage opening is not so severely loaded. The design seismic load on the Wall Line A is 745 lb. Using the approach above (and substituting a safety factor of 2.5 for seismic design), the value of $F_{s,ext}$ determined is 905 plf which is much less than the 2,004 plf determined for the design wind shear load condition assumed in this example. By inspecting Table 6.1, 7/16-inch-thick Structural 1 sheathing is sufficient and the pnuematic nails used on the rest of the building's shear walls may be used. However, this requires the two garage return walls to be restrained with two hold-down brackets each as in the segmented shear wall design method. For the seismic load, the garage opening wall (Wall Line A) may be suitably designed as a perforated shear wall and eliminate the need for two of the four hold-downs. A portal frame may also be considered for the garage opening (see Section 6.5.2.7).

Wall Line D/E may be designed in a similar fashion to the options discussed above. In fact, Wall Line E may be eliminated as a designed shear wall line provided that a collector is provided to bring the diaphragm shear load into the single wall segment in wall line D (see the dotted line on the floor plan figure). Of course, Wall line D must be designed to carry the full design shear load assigned to that end of the building. Collector design was illustrated in Example 6.3. The connections for overturning (i.e., hold-downs) and base shear transfer must be designed as illustrated in Examples 6.1 and 6.2. As an additional option, Wall Line C may be designed as an interior shear wall line and the wood structural panel sheathing would be placed underneath the interior finish. This last option would relieve some of the load on the house end walls and possibly simplify the overall shear wall construction details used in the house.

3. Determine the shear loads on the N-S shear wall lines using the relative stiffness method and an assumed shear wall construction for the given seismic design condition only.

Assume that the shear wall construction will be as follows:

- 7/16-inch OSB Structural I wood structural panel sheathing with 8d common nails (or 0.131-inch diameter 8d pneumatic nails) spaced at 4 inches on center on the panel edges and 12 inches in the panel field.
- Douglas-fir wall framing is used with 2x studs spaced at 16 inches on center.
- Walls are designed as perforated shear wall lines and adequate hold-downs and base shear connections are provided.

It will be further assumed that the house and garage are sufficiently tied together to act as a structural unit. It must be remembered that the relative stiffness design approach is predicated on the assumption that the horizontal diaphragm is rigid in comparison to the supporting shear walls so that the forces are distributed according to the relative stiffness of the shear wall lines. This assumption is exactly opposite to that assumed by use of the tributary area method.

As given for the design example, the following design seismic shear loads apply to the first story of the example building:

Design N-S Seismic Lateral Load (mapped S_s = 1.5g)

House: 7,493 lb total story shear (tributary weight is 37,464 lb)
Garage: 1,490 lb total story shear (tributary weight is 7,452 lb)
Total: 8,983 lb total story shear (total tributary weight is 44,916 lb)

Locate the center of gravity

The first step is to determine the center of gravity of the building at the first story level. The total seismic story shear load will act through this point. For wind design, the process is similar, but the horizontal wind forces on various portions of the building (based on vertical projected areas and wind pressures) are used to determine the force center for the lateral wind loads (i.e., the resultant of the garage and house lateral wind loads).

Establishing the origin of an x-y coordinate system at the bottom corner of Wall Line B of the example first floor plan, the location of the center of gravity is determined by taking weighted moments about each coordinate axis using the center of gravity location for the garage and house portions. Again, the "bump-out" area in living room is considered to have negligible impact on the estimate of the center of gravity since most of the building mass is originating from the second story and roof which does not have the "bump-out" in the plan.

The center of gravity of the garage has the (x,y) coordinates of (-11 ft, 16 ft). The center of gravity of the house has the coordinates (21 ft, 14 ft).

Weighted moments about the y-axis:

$X_{cg,building}$ = $[(X_{cg,garage})$(garage weight) + $(X_{cg,house})$(house weight)]/(total weight)
= [(-11 ft)(7,452 lb) + (21 ft)(37,464 lb)]/(44,916 lb)
= 15.7 ft

Weighted moments about the x-axis:

$Y_{cg,building}$ = $[(Y_{cg,garage})$(garage weight) + $(Y_{cg,house})$(house weight)]/(total weight)
= [(16 ft)(7,452 lb) + (14 ft)(37,464 lb)]/(44,916 lb)
= 14.3 ft

Thus, the center of gravity for the first story is located at the (x,y) coordinates of (15.7 ft, 14.3 ft). The approximate location on the floor plan is about 4 inches north of the center bearing wall line and directly in front of the stair well leading down (i.e., about 5 feet to the left of the center of the house).

Locate the center of resistance

The center of resistance is somewhat more complicated to determine and requires an assumption regarding the shear wall stiffness. Two methods of estimating the relative stiffness of segmented shear walls are generally recognized. One method bases the segmented shear wall stiffness on it's length. Thus, longer shear walls have greater stiffness (and capacity). However, this method is less appealing when multiple segments are included in one wall line and particularly when the segments have varying aspect ratios, especially narrow aspect ratios which affect stiffness disproportionately to the length. The second method bases the segmented shear wall stiffness on the shear capacity of the segment, which is more appealing when various shear wall constructions are used with variable unit shear values and when variable aspect ratios are used, particularly when the unit shear strength is corrected for narrow aspect ratios. The method based on strength is also appropriate for use with the perforated shear wall method, since the length of a perforated shear wall has little to do with its stiffness or strength. Rather, the amount of openings in the wall (as well as its construction) govern its stiffness and capacity. Therefore, the method used in this example will use the capacity of the perforated shear wall lines as a measure of relative stiffness. The same technique may be used with a segmented shear wall design method by determining the shear capacity of each shear wall line (comprised of one or more shear wall segments) as shown in Example 6.1.

First, the strength of each shear wall line in the building must be determined. Using the perforated shear wall method and the assumed wall construction given at the beginning of Step 3, the design shear wall line capacities (see below) are determined for each of the exterior shear wall lines in the building. The window and door opening sizes are shown on the plan so that the perforated shear wall calculations can be done as demonstrated in Example 6.2. It is assumed that no interior shear wall lines will be used (except at the shared wall between the garage and the house) and that the contribution of the interior partition walls to the stiffness of the building is negligible. As mentioned, this assumption can overlook a significant factor in the lateral resistance and stiffness of a typical residential building.

PSW 1: F_{psw1} = 7,812 lb (Wall Line D)
PSW 2: F_{psw2} = 3,046 lb (Wall Line E)
PSW 3: F_{psw3} = 14,463 lb (North side wall of house)
PSW 4: F_{psw4} = 9,453 lb (North side of garage)
PSW 5: F_{psw5} = 182 lb (Wall Line A; garage opening)
PSW 6: F_{psw6} = 9,453 lb (South side wall of garage)
PSW 7: F_{psw7} = 9,687 lb (Wall Line B)
PSW 8: F_{psw8} = 11,015 lb (South side wall of house at front)

The center of stiffness on the y-coordinate is now determined as follows using the above PSW design shear capacities for wall lines oriented in the E-W direction:

$$Y_{cs} = [(F_{psw3})(Y_{psw3}) + (F_{psw4})(Y_{psw4}) + (F_{psw6})(Y_{psw6}) + (F_{psw8})(Y_{psw8})]/(F_{psw,E\text{-}W})$$
$$= [(14{,}463 \text{ lb})(28 \text{ ft})+(9{,}453 \text{ lb})(26 \text{ ft})+(9{,}453 \text{ lb})(6 \text{ ft})+(11{,}015 \text{ lb})(0 \text{ ft})]/(44{,}384 \text{ lb})$$
$$= 15.9 \text{ ft}$$

The center of stiffness on the x-coordinate is determined similarly considering the wall lines oriented in the N-S direction:

$$X_{cs} = [(F_{psw1})(X_{psw1}) + (F_{psw2})(X_{psw2}) + (F_{psw5})(X_{psw5}) + (F_{psw7})(X_{psw7})]/(F_{psw,N\text{-}S})$$
$$= [(7{,}812 \text{ lb})(42 \text{ ft})+(3{,}046 \text{ lb})(48 \text{ ft})+(182 \text{ lb})(-22 \text{ ft})+(9{,}687 \text{ lb})(0 \text{ ft})]/(20{,}727 \text{ lb})$$
$$= 22.7 \text{ ft}$$

Therefore, the coordinates of the center of stiffness are (22.7 ft, 15.9 ft). Thus, the center of stiffness is located to the right of the center of gravity (force center for the seismic load) by 22.7 ft – 15.7 ft = 7 ft. This offset between the center of gravity and the center of resistance will create a torsional response in the N-S seismic load direction under consideration. For E-W seismic load direction, the offset (in the y-coordinate direction) is only 15.9 ft – 14.3 ft = 1.6 ft which is practically negligible from the standpoint of torsional response. It should be remembered that, in both loading directions, the influence of interior partitions on the center of stiffness (and thus the influence on torsional response) is not considered. To conservatively account for this condition and for possible error in locating the actual center of gravity of the building (i.e., accidental torsion), codes usually require that the distance between the center of gravity and the center of stiffness be considered as a minimum of 5 percent of the building dimension perpendicular to the direction of seismic force under consideration. This condition is essentially met in this example since the offset dimension for the N-S load direction is 7 feet which is 10 percent of the E-W plan dimension of the house and attached garage.

Distribute the direct shear forces to N-S walls

The direct shear force is distributed to the N-S walls based on their relative stiffness without regard to the location of the center of stiffness (resistance) and the center of gravity (seismic force center), or the torsional load distribution that occurs when they are offset from each other. The torsional load distribution is superimposed on the direct shear forces on the shear wall lines in the next step of the process.

The direct seismic shear force of 8,983 lb is distributed as shown below based on the relative stiffness of the perforated shear wall lines in the N-S direction. As before, the relative stiffness is based on the design shear capacity of each perforated shear wall line relative to that of the total design capacity of the N-S shear wall lines.

Direct shear on PSW1, PSW2, PSW5, and PSW7 is determined as follows:

(total seismic shear load on story)$[(F_{psw1})/(F_{psw,N\text{-}S})]$ = (8,983 lb)[(7,812 lb)/(20,727 lb)]
= (8,983 lb)[0.377]
= 3,387 lb

(total seismic shear load on story)$[(F_{psw2})/(F_{psw,N\text{-}S})]$ = (8,983 lb)[(3,046 lb)/(20,727 lb)]
= (8,983 lb)[0.147]
= 1,321 lb

(total seismic shear load on story)$[(F_{psw5})/(F_{psw,N\text{-}S})]$ = (8,983 lb)[(182 lb)/(20,727 lb)]
= (8,983 lb)[0.009]
= 81 lb

(total seismic shear load on story)$[(F_{psw7})/(F_{psw,N\text{-}S})]$ = (8,983 lb)[(9,687 lb)/(20,727 lb)]
= (8,983 lb)[0.467]
= 4,195 lb

Distribute the torsion load

The torsional moment is created by the offset of the center of gravity (seismic force center) from the center of stiffness or resistance (also called the center of rigidity). For the N-S load direction, the torsional moment is equal to the total seismic shear load on the story multiplied by the x-coordinate offset of the center of gravity and the center of stiffness (i.e., 8,983 lb x 7 ft = 62,881 ft-lb). The sharing of this torsional moment on all of the shear wall lines is based on the torsional moment of resistance of each wall line. The torsional moment of resistance is determined by the design shear capacity of each wall line (used as the measure of relative stiffness) multiplied by the square of its distance from the center of stiffness. The amount of the torsional shear load (torsional moment) distributed to each wall line is then determined by the each wall's torsional moment of resistance in proportion to the total torsional moment of resistance of all shear wall lines combined. The torsional moment of resistance of each shear wall line and the total for all shear wall lines (torsional moment of inertia) is determined as shown below.

Wall Line	F_{psw}	Distance from Center of Resistance	$F_{psw}(d)^2$
PSW1	7,812 lb	19.3 ft	2.91×10^6 lb-ft^2
PSW2	3,046 lb	25.3 ft	1.95×10^6 lb-ft^2
PSW3	14,463 lb	12.1 ft	2.12×10^6 lb-ft^2
PSW4	9,453 lb	10.1 ft	9.64×10^5 lb-ft^2
PSW5	182 lb	44.7 ft	3.64×10^5 lb-ft^2
PSW6	9,453 lb	9.9 ft	9.26×10^5 lb-ft^2
PSW7	9,687 lb	22.7 ft	4.99×10^6 lb-ft^2
PSW8	11,015 lb	15.9 ft	2.78×10^6 lb-ft^2
Total torsional moment of inertia (J)			1.70×10^7 lb-ft^2

Now, the torsional shear load on each wall is determined using the following basic equation for torsion:

$$V_{WALL} = \frac{M_T d(F_{WALL})}{J}$$

where,

V_{WALL} = the torsional shear load on the wall line (lb)
M_T = the torsional moment* (lb-ft)
d = the distance of the wall from the center of stiffness (ft)
F_{WALL} = the design shear capacity of the segmented or perforated shear wall line (lb)
J = the torsional moment of inertia for the story (lb-ft^2)

*The torsional moment is determined by multiplying the design shear load on the story by the offset of the center of stiffness relative to the center of gravity perpendicular to the load direction under consideration. For wind design, the center of the vertical projected area of the building is used in lieu of the center gravity.

Now, the torsional loads may be determined as shown below for the N-S and E-W wall lines. For PSW1 and PSW2 the torsion load is in the reverse direction of the direct shear load on these walls. This behavior is the result of the center of shear resistance being offset from the force center which causes rotation about the center of stiffness. (Center of shear resistance and center of stiffness may be used interchangeably since the shear resistance is assumed to represent stiffness.) If the estimated offset of the center of gravity and the center of stiffness is reasonably correct, then the torsional response will tend to reduce the shear load on PSW1 and PSW2. However, codes generally do not allow the direct shear load on a wall line to be reduced due to torsion – only increases should be considered.

The following values for use in the torsion equation apply to this example:

M_T = (8,983 lb)(7 ft) = 62,881 ft-lb
J = 1.70 x 10^7 lb-ft^2

The torsional loads on PSW5 and PSW7 are determined as follows:

V_{psw5} = (62,881 ft-lb)(44.7 ft)(182 lb) / (1.70 x 10^7 lb-ft^2)
= 30 lb

V_{psw7} = (62,881 ft-lb)(22.7 ft)(9,687 lb) / (1.70 x 10^7 lb-ft^2)
= 813 lb

These torsional shear loads are added to the direct shear loads for the N-S walls and the total design shear load on each wall line may be compared to its design shear capacity as shown below.

N-S Wall Lines	Wall Design Capacity, F_{psw} (lb)	Direct Shear Load (lb)	Torsional Shear Load (lb)	Total Design Shear Load (lb)	Percent of Design Capacity Used
PSW1	7,812	3,387	na*	3,387	43% (ok)
PSW2	3,046	1,321	na*	1,321	43% (ok)
PSW5	182	81	30	111	61% (ok)
PSW7	9,687	4,195	813	5,008	52% (ok)

*The torsional shear load is actually in the reverse direction of the direct shear load for these walls, but it is not subtracted as required by code practice.

While all of the N-S shear wall lines have sufficient design capacity, it is noticeable that the wall lines on the left side (West) of the building are "working harder" and the walls on the right side (East) of the building are substantially over-designed. The wall construction could be changed to allow a greater sheathing nail spacing on walls PSW1 and PSW2. Also, the assumption of a rigid diaphragm over the entire expanse of the story is very questionable, even if the garage is "rigidly" tied to the house with adequate connections. It is likely that the loads on Walls PSW5 and PSW7 will be higher than predicted using the relative stiffness method. Thus, the tributary area method (see Step 2) may provide a more reliable design and should be considered along with the above analysis. Certainly, reducing the shear wall construction based on the above analysis is not recommended prior to "viewing" the design from the perspective of the tributary area approach. Similarly, the garage opening wall (PSW5) should not be assumed to be adequate simply based on the above analysis in view of the inherent assumptions of the relative stiffness method in the horizontal distribution of shear forces. For more compact buildings with continuous horizontal diaphragms extending over the entire area of each story, the method is less presumptive in nature. But, this qualitative observation is true of all of the force distribution methods demonstrated in this design example.

Conclusion

This seemingly simple design example has demonstrated the many decisions, variables, and assumptions to consider in designing the lateral resistance of a light-frame home. For an experienced designer, certain options or standardized solutions may become favored and developed for repeated use in similar conditions. Also, an experienced designer may be able to effectively design using simplified analytical methods (i.e. the total shear approach shown in Step 1) supplemented with judgment and detailed evaluations of certain portions or unique details as appropriate.

In this example, it appears that a 7/16-inch-thick Structural I wood structural panel sheathing can be used for all shear wall construction to resist the required wind shear loading. A constant sheathing panel edge nail spacing is also possible by using 3 inches on center if the perforated shear wall method is used and 4 inches on center if the segmented shear wall method is used (based on the worst-case condition of Wall Line B). The wall sheathing nails specified were 8d pneumatic nails with a 0.113 inch diameter. In general, this wall construction will be conservative for most wall lines on the first story of the example house. If the seismic shear load were the only factor (i.e., the wind load condition was substantially less than assumed), the wall construction could be simplified even more such that a perforated shear wall design approach with a single sheathing fastening requirement may be suitable for all shear wall lines. The garage opening wall would be the only exception.

Finally, numerous variations in construction detailing in a single project should be avoided as it may lead to confusion and error in the field. Fewer changes in assembly requirements, fewer parts, and fewer special details should all be as important to the design objectives as meeting the required design loads. When the final calculation is done (regardless of the complexity or simplicity of the analytic approach chosen and the associated uncertainties or assumptions), the designer should exercise judgment in making reasonable final adjustments to the design to achieve a practical, well-balanced design. As a critical final consideration, the designer should be confident that the various parts of the structural system are adequately “tied together” to act as a structural unit in resisting the lateral loads. This consideration is as much a matter of judgement as it is a matter of analysis.

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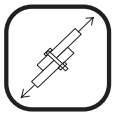


CHAPTER 7

Connections

7.1 General

The objectives of connection design are

- to transfer loads resisted by structural members and systems to other parts of the structure to form a "continuous load path";
- to secure nonstructural components and equipment to the building; and
- to fasten members in place during construction to resist temporary loads during installation (i.e., finishes, sheathing, etc.).

Adequate connection of the framing members and structural systems covered in Chapters 4, 5, and 6 is a critical design and construction consideration. Regardless of the type of structure or type of material, structures are only as strong as their connections, and structural systems can behave as a unit only with proper interconnection of the components and assemblies; therefore, this chapter is dedicated to connections. A connection transfers loads from one framing member to another (i.e., a stud to a top or bottom plate) or from one assembly to another (i.e., a roof to a wall, a wall to a floor, and a floor to a foundation). Connections generally consist of two or more framing members and a mechanical connection device such as a fastener or specialty connection hardware. Adhesives are also used to supplement mechanical attachment of wall finishes or floor sheathing to wood.

This chapter focuses on conventional wood connections that typically use nails, bolts, and some specialty hardware. The procedures for designing connections are based on the *National Design Specification for Wood Construction* (NDS) (AF&PA, 1997). The chapter also addresses relevant concrete and masonry connections in accordance with the applicable provisions of *Building Code Requirements for Structural Concrete* (ACI-318) and *Building Code Requirements for Masonry Structures* (ACI-530)(ACI, 1999a; ACI 1999b). When referring to the NDS, ACI-318, or ACI-530, the chapter identifies particular sections as NDS•12.1, ACI-318•22.5, or ACI-530•5.12.

For most connections in typical residential construction, the connection design may be based on prescriptive tables found in the applicable residential building code (ICC, 1998). Table 7.1 depicts a commonly recommended nailing schedule for wood-framed homes.

TABLE 7.1 Recommended Nailing Schedule for a Wood-Framed Home[1]

Application	Nailing Method	Number of Nails	Size of Nail	Notes
Header to joist	End-nail	3	16d	
Joist to sill or girder	Toenail	2	10d	
	Toenail	3	8d	
Header and stringer (band) joists to sill	Toenail		8d	16 inches on center
Board sheathing	Face-nail	2 or 3	8d	To each joist
Stud to sole plate or top plate	End-nail	2	16d	At each stud
	Toenail	4	8d	
Sole plate to joist or blocking	Face-nail		16d	16 inches on center
Doubled studs	Face-nail, stagger		10d	16 inches on center
End stud of interior wall to exterior wall stud	Face-nail		16d	16 inches on center
Upper top plate to lower top plate	Face-nail		10d	16 inches on center
Double top plate, laps and intersections	Face-nail	4	10d	
Continuous header, two pieces, each edge	Face-nail		10d	12 inches on center
Ceiling joist to top wall plates	Toenail	3	8d	
Ceiling joist laps at partition	Face-nail	4	16d	
Rafter to top plate	Toenail	3	8d	
Rafter to ceiling joist	Face-nail	4	16d	
Rafter to valley or hip rafter	Toenail	4	10d	
Rafter to ridge board	Endnail	3	16d	
	Toenail	4	8d	
Collar beam to rafter, 2-inch member	Face-nail	2	12d	
Collar beam to rafter, 1-inch member	Face-nail	3	8d	
Diagonal let-in brace to each stud and plate, 1-inch member	Face-nail	2	8d	
Intersecting studs at corners	Face-nail		16d	12 inches on center
Built-up girder and beams, three or more members, each edge	Face-nail		10d	12 inches on center each ply
Maximum 1/2-inch-thick (or less) wood structural panel wall sheathing	Face-nail	6d at 6 inches on center at panel edges; 12 inches on center at intermediate framing		
Minimum 1/2-inch-thick (or greater) wood structural panel wall/roof/floor sheathing	Face-nail	8d at 6 inches on center at panel edges; 12 inches on center at intermediate framing		
Wood sill plate to concrete or masonry		1/2-inch-diameter anchor bolt at 6 feet on center and within 1 foot from ends of sill members		

Source: *Based on current industry practice and other sources (ICC,1998, NAHB, 1994; NAHB, 1982).*
Note:
[1]In practice, types of nails include common, sinker, box, or pneumatic; refer to Section 7.2 for descriptions of these fasteners. Some recent codes have specified that common nails are to be used in all cases. However, certain connections may not necessarily require such a nail or may actually be weakened by use of a nail that has too large a diameter (i.e., causing splitting of wood members). Other codes allow box nails to be used in most or all cases. NER-272 guidelines for pneumatic fasteners should be consulted (NES, Inc., 1997). However, the NER-272 guidelines are based on simple, conservative conversions of various code nail schedules, such as above, using the assumption that the required performance is defined by a common nail in all applications. In short, there is a general state of confusion regarding appropriate nailing requirements for the multitude of connections and related purposes in conventional residential construction.

The NDS recognizes in NDS•7.1.1.4 that "extensive experience" constitutes a reasonable basis for design; therefore, the designer may use Table 7.1 for many, if not all, connections. However, the designer should consider carefully the footnote to Table 7.1 and verify that the connection complies with local requirements, practice, and design conditions for residential construction. A connection design based on the NDS or other sources may be necessary for special conditions such as high-hazard seismic or wind areas and when unique structural details or materials are used.

In addition to the conventional fasteners mentioned above, many specialty connectors and fasteners are available on today's market. The reader is encouraged to gather, study, and scrutinize manufacturer literature regarding specialty fasteners, connectors, and tools that meet a wide range of connection needs.

7.2 Types of Mechanical Fasteners

Mechanical fasteners that are generally used for wood-framed house design and construction include the following:

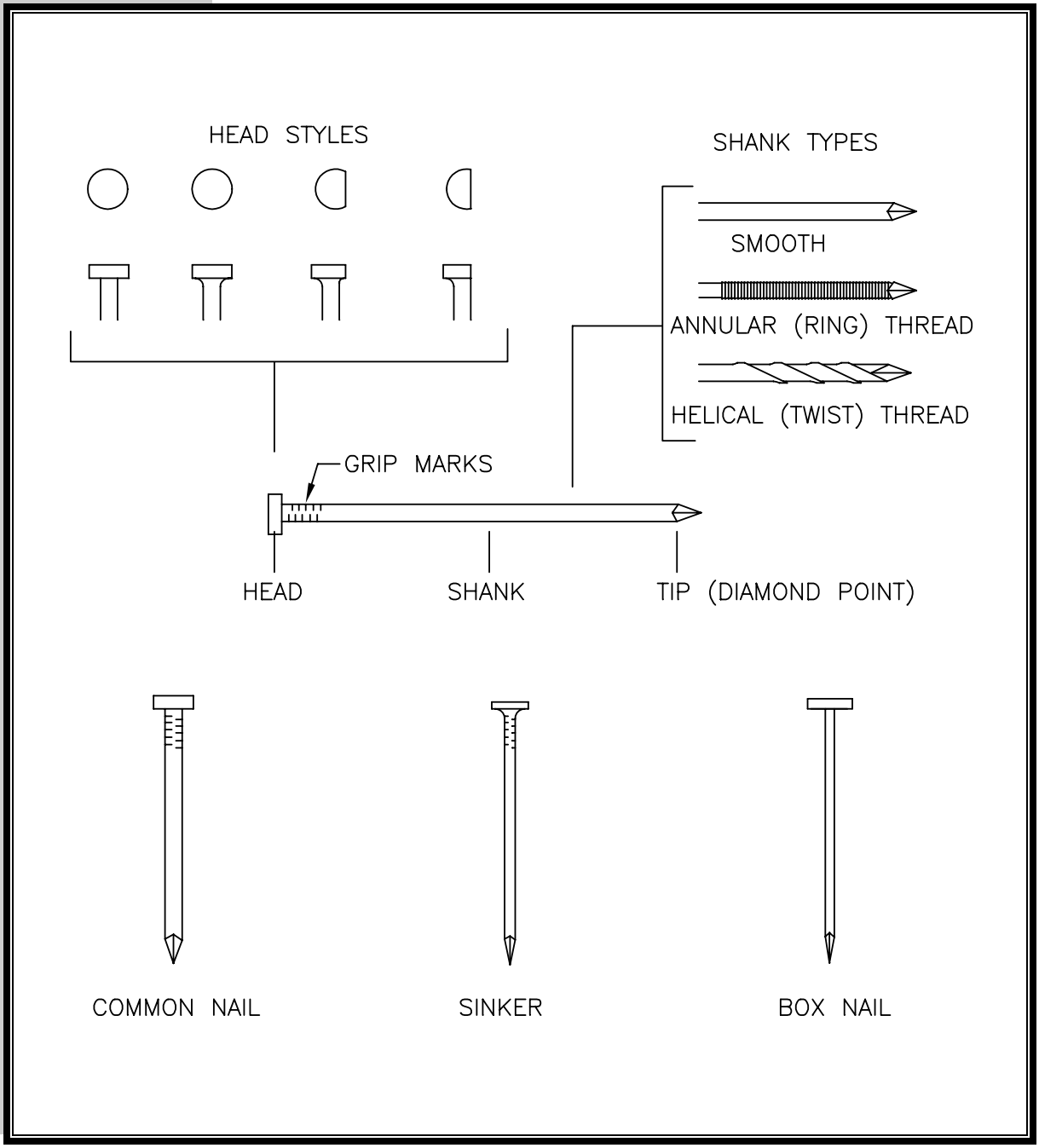
- nails and spikes;
- bolts;
- lag bolts (lag screws); and
- specialty connection hardware.

This section presents some basic descriptions and technical information on the above fasteners. Sections 7.3 and 7.4 provide design values and related guidance. Design examples are provided in Section 7.5 for various typical conditions in residential wood framing and foundation construction.

7.2.1 Nails

Several characteristics distinguish one nail from another. Figure 7.1 depicts key nail features for a few types of nails that are essential to wood-framed design and construction. This section discusses some of a nail's characteristics relative to structural design; the reader is referred to *Standard Terminology of Nails for Use with Wood and Wood-Base Materials* (ASTM F547) and *Standard Specification for Driven Fasteners: Nails, Spikes, and Staples* (ASTM F 1667) for additional information (ASTM, 1990; ASTM, 1995).

FIGURE 7.1 Elements of a Nail and Nail Types



The most common nail types used in residential wood construction follow:

- *Common nails* are bright, plain-shank nails with a flat head and diamond point. The diameter of a common nail is larger than that of sinkers and box nails of the same length. Common nails are used primarily for rough framing.

- *Sinker nails* are bright or coated slender nails with a sinker head and diamond point. The diameter of the head is smaller than that of a common nail with the same designation. Sinker nails are used primarily for rough framing and applications where lumber splitting may be a concern.
- *Box nails* are bright, coated, or galvanized nails with a flat head and diamond point. They are made of lighter-gauge wire than common nails and sinkers and are commonly used for toenailing and many other light framing connections where splitting of lumber is a concern.
- *Cooler nails* are generally similar to the nails above, but with slightly thinner shanks. They are commonly supplied with ring shanks (i.e., annular threads) as a drywall nail.
- *Power-driven nails* (and staples) are produced by a variety of manufacturers for several types of power-driven fasteners. Pneumatic-driven nails and staples are the most popular power-driven fasteners in residential construction. Nails are available in a variety of diameters, lengths, and head styles. The shanks are generally cement-coated and are available with deformed shanks for added capacity. Staples are also available in a variety of wire diameters, crown widths, and leg lengths. Refer to NER-272 for additional information and design data (NES, Inc., 1997).

Nail lengths and weights are denoted by the *penny weight*, which is indicated by *d*. Given the standardization of common nails, sinkers, and cooler nails, the penny weight also denotes a nail's head and shank diameter. For other nail types, sizes are based on the nail's length and diameter. Table 7.2 arrays dimensions for the nails discussed above. The nail length and diameter are key factors in determining the strength of nailed connections in wood framing. The steel yield strength of the nail may also be important for certain shear connections, yet such information is rarely available for a "standard" lot of nails.

TABLE 7.2 Nail Types, Sizes, and Dimensions[1]

Type of Nail	Nominal Size (penny weight, d)	Length (inches)	Diameter (inches)
Common	6d	2	0.113
	8d	2 1/2	0.131
	10d	3	0.148
	12d	3 1/4	0.148
	16d	3 1/2	0.162
	20d	4	0.192
Box	6d	2	0.099
	8d	2 12	0.113
	10d	3	0.128
	12d	3 1/4	0.128
	16d	3 1/2	0.135
Sinker	6d	1 7/8	0.092
	8d	2 3/8	0.113
	10d	2 7/8	0.120
	12d	3 1/8	0.135
	16d	3 1/4	0.148
Pneumatic[2]	6d	1 7/8 to 2	0.092 to 0.113
	8d	2 3/8 to 2 1/2	0.092 to 0.131
	10d	3	0.120 to 0.148
	12d	3 1/4	0.120 to 0.131
	16d	3 1/2	0.131 to 0.162
	20d	4	0.131
Cooler	4d	1 3/8	0.067
	5d	1 5/8	0.080
	6d	1 7/8	0.092

Notes
[1]Based on ASTM F 1667 (ASTM, 1995).
[2]Based on a survey of pneumatic fastener manufacturer data and NER-272 (NES, Inc., 1997).

There are many types of *nail heads*, although three types are most commonly used in residential wood framing.

- The *flat nail head* is the most common head. It is flat and circular, and its top and bearing surfaces are parallel but with slightly rounded edges.
- The *sinker nail head* is slightly smaller in diameter than the flat nail head. It also has a flat top surface; however, the bearing surface of the nail head is angled, allowing the head to be slightly countersunk.
- *Pneumatic nail heads* are available in the above types; however, other head types such as a half-round or D-shaped heads are also common.

The *shank*, as illustrated in Figure 7.1, is the main body of a nail. It extends from the head of the nail to the point. It may be plain or deformed. A plain shank is considered a "smooth" shank, but it may have "grip marks" from the manufacturing process. A deformed shank is most often either threaded or fluted to provide additional withdrawal or pullout resistance. Threads are annular

(i.e., ring shank), helical, or longitudinal deformations rolled onto the shank, creating ridges and depressions. Flutes are helical or vertical deformations rolled onto the shank. Threaded nails are most often used to connect wood to wood while fluted nails are used to connect wood to concrete (i.e., sill plate to concrete slab or furring strip to concrete or masonry). Shank diameter and surface condition both affect a nail's capacity.

The *nail tip*, as illustrated in Figure 7.1, is the end of the shank–usually tapered–that is formed during manufacturing to expedite nail driving into a given material. Among the many types of nail points, the *diamond point* is most commonly used in residential wood construction. The diamond point is a symmetrical point with four approximately equal beveled sides that form a pyramid shape. A *cut point* used for concrete cut nails describes a blunt point. The point type can affect nail drivability, lumber splitting, and strength characteristics.

The *material* used to manufacture nails may be steel, stainless steel, heat-treated steel, aluminum, or copper, although the most commonly used materials are steel, stainless steel, and heat-treated steel. *Steel* nails are typically formed from basic steel wire. *Stainless steel* nails are often recommended in exposed construction near the coast or for certain applications such as cedar siding to prevent staining. Stainless steel nails are also recommended for permanent wood foundations. *Heat-treated* steel includes annealed, case-hardened, or hardened nails that can be driven into particularly hard materials such as extremely dense wood or concrete.

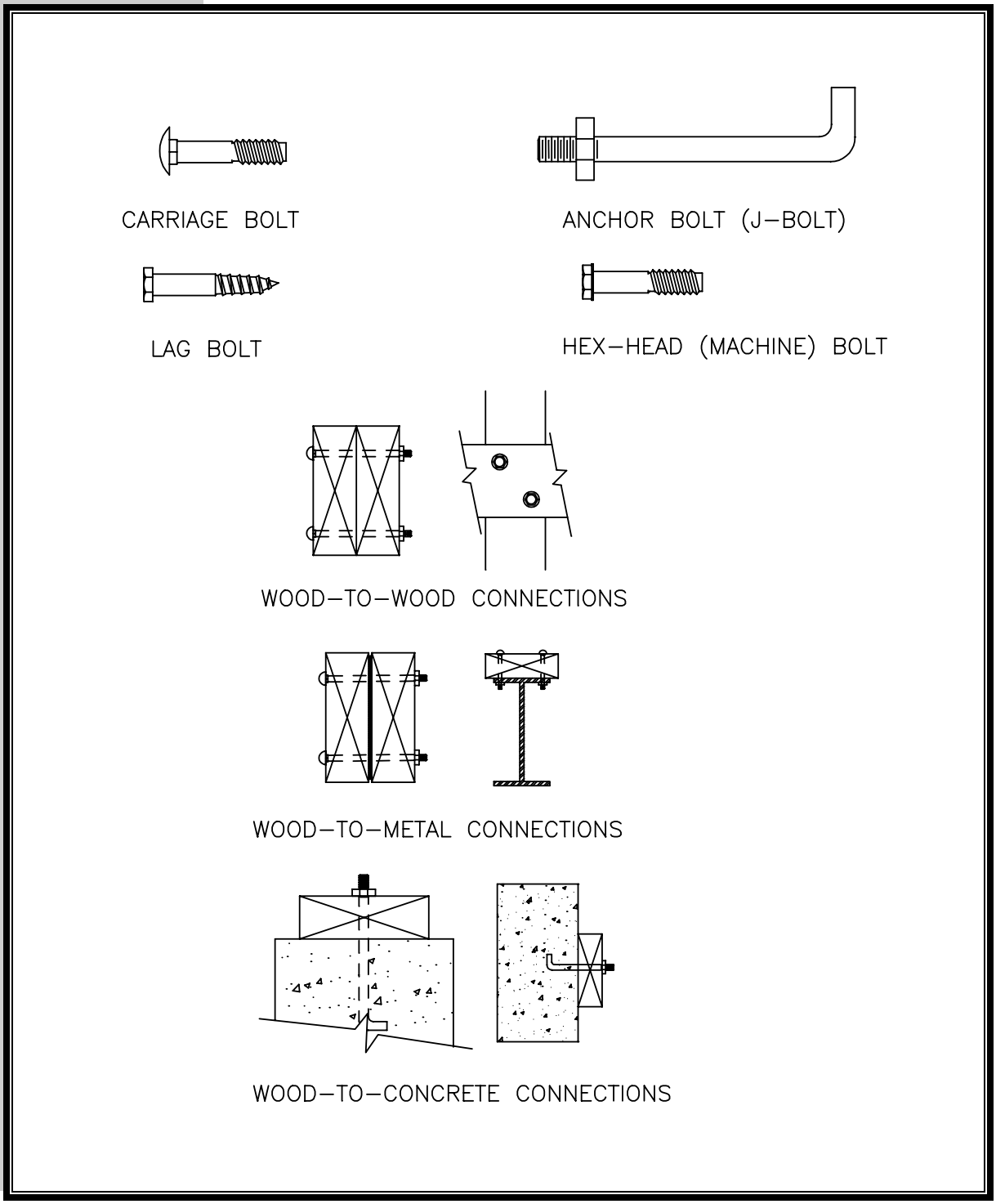
Various nail *coatings* provide corrosion resistance, increased pullout resistance, or ease of driving. Some of the more common coatings in residential wood construction are described below.

- *Bright*. Uncoated and clean nail surface.
- *Cement-coated*. Coated with a heat-sensitive cement that prevents corrosion during storage and improves withdrawal strength depending on the moisture and density of the lumber and other factors.
- *Galvanized*. Coated with zinc by barrel-tumbling, dipping, electroplating, flaking, or hot-dipping to provide a corrosion-resistant coating during storage and after installation for either performance or appearance. The coating thickness increases the diameter of the nail and improves withdrawal and shear strength.

7.2.2 Bolts

Bolts are often used for "heavy" connections and to secure wood to other materials such as steel or concrete. In many construction applications, however, special power-driven fasteners are used in place of bolts. Refer to Figure 7.2 for an illustration of some typical bolt types and connections for residential use.

FIGURE 7.2 Bolt and Connection Types



In residential wood construction, bolted connections are typically limited to wood-to-concrete connections unless a home is constructed in a high-hazard wind or seismic area and hold-down brackets are required to transfer shear wall overturning forces (see Chapter 6). Foundation bolts, typically embedded in concrete or grouted masonry, are commonly referred to as *anchor bolts, J-bolts,*

or *mud-sill anchors.* Another type of bolt sometimes used in residential construction is the *structural bolt,* which connects wood to steel or wood to wood. Low-strength ASTM A307 bolts are commonly used in residential construction as opposed to high-strength ASTM A325 bolts, which are more common in commercial applications. Bolt diameters in residential construction generally range from 1/4- to 3/4-inch, although 1/2- to 5/8-inch-diameter bolts are most common, particularly for connecting a 2x wood sill to grouted masonry or concrete.

Bolts, unlike nails, are installed in predrilled holes. If holes are too small, the possibility of splitting the wood member increases during installation of the bolt. If bored too large, the bolt holes encourage nonuniform dowel (bolt) bearing stresses and slippage of the joint when loaded. NDS•8.1 specifies that bolt holes should range from 1/32- to 1/16-inch larger than the bolt diameter to prevent splitting and to ensure reasonably uniform dowel bearing stresses.

7.2.3 Specialty Connection Hardware

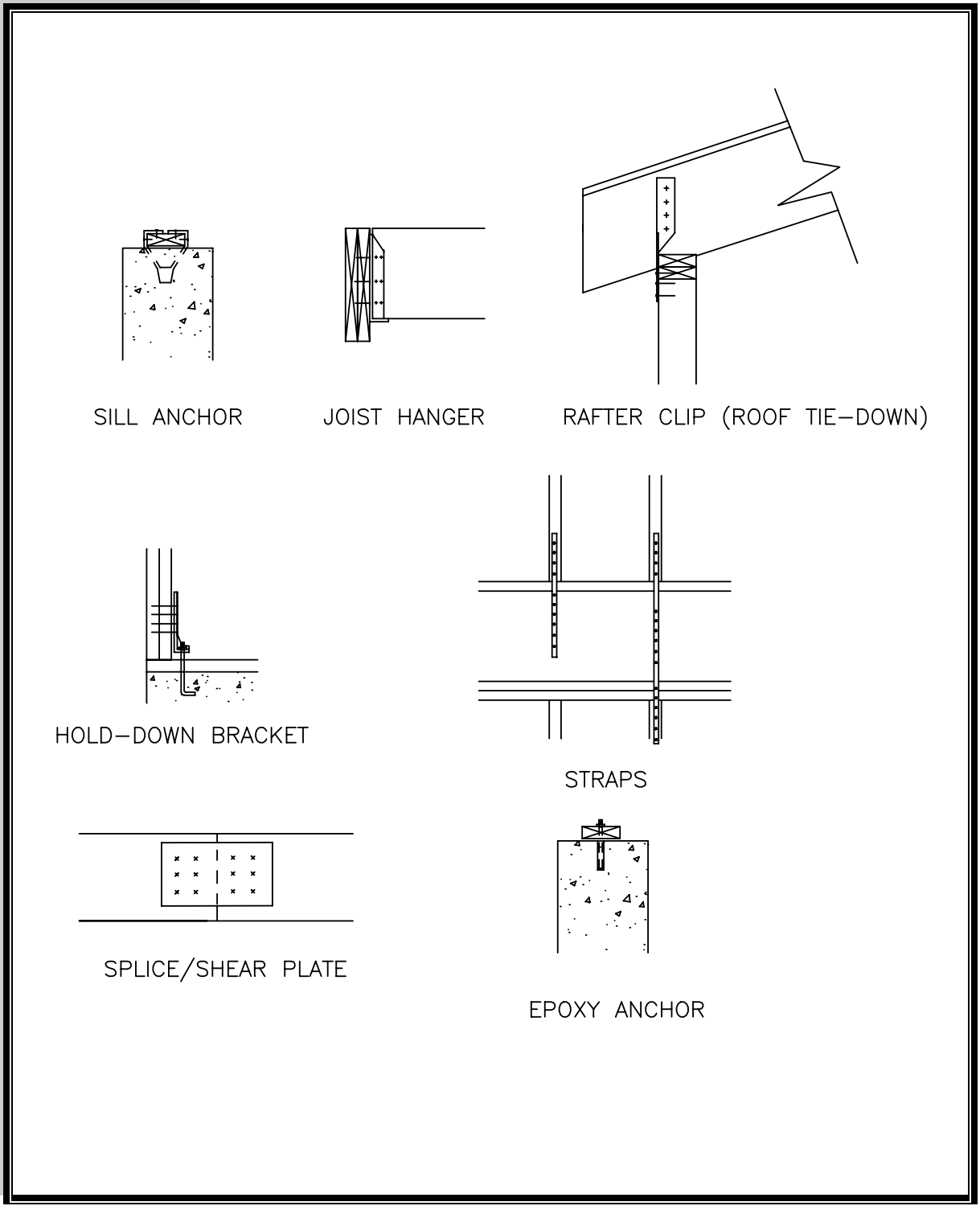
Many manufacturers fabricate specialty connection hardware. The load capacity of a specialty connector is usually obtained through testing to determine the required structural design values. The manufacturer's product catalogue typically provides the required values. Thus, the designer can select a standard connector based on the design load determined for a particular joint or connection (see Chapter 3). However, the designer should carefully consider the type of fastener to be used with the connector; sometimes a manufacturer requires or offers proprietary nails, screws, or other devices. It is also recommended that the designer verify the safety factor and strength adjustments used by the manufacturer, including the basis of the design value. In some cases, as with nailed and bolted connections in the NDS, the basis is a serviceability limit state (i.e., slip or deformation) and not ultimate capacity.

A few examples of specialty connection hardware are illustrated in Figure 7.3 and discussed below.

- *Sill anchors* are used in lieu of foundation anchor bolts. Many configurations are available in addition to the one shown in Figure 7.3.
- *Joist hangers* are used to attach single or multiple joists to the side of girders or header joists.
- *Rafter clips* and *roof tie-downs* are straps or brackets that connect roof framing members to wall framing to resist roof uplift loads associated with high-wind conditions.
- *Hold-down brackets* are brackets that are bolted, nailed, or screwed to wall studs or posts and anchored to the construction below (i.e., concrete, masonry, or wood) to "hold down" the end of a member or assembly (i.e., shear wall).
- *Strap ties* are prepunched straps or coils of strapping that are used for a variety of connections to transfer tension loads.
- *Splice plates* or *shear plates* are flat plates with prepunched holes for fasteners to transfer shear or tension forces across a joint.

- *Epoxy-set anchors* are anchor bolts that are drilled and installed with epoxy adhesives into concrete after the concrete has cured and sometimes after the framing is complete so that the required anchor location is obvious.

FIGURE 7.3 Specialty Connector Hardware



7.2.4 Lag Screws

Lag screws are available in the same diameter range as bolts; the principal difference between the two types of connectors is that a lag screw has screw threads that taper to a point. The threaded portion of the lag screw anchors itself in the main member that receives the tip. Lag screws (often called lag bolts) function as bolts in joints where the main member is too thick to be economically penetrated by regular bolts. They are also used when one face of the member is not accessible for a "through-bolt." Holes for lag screws must be carefully drilled to one diameter and depth for the shank of the lag screw and to a smaller diameter for the threaded portion. Lag screws in residential applications are generally small in diameter and may be used to attach garage door tracks to wood framing, steel angles to wood framing supporting brick veneer over wall openings, various brackets or steel members to wood, and wood ledgers to wall framing.

7.3 Wood Connection Design

7.3.1 General

This section covers the NDS design procedures for nails, bolts, and lag screws. The procedures are intended for allowable stress design (ASD) such that loads should be determined accordingly (see Chapter 3). Other types of fastenings are addressed by the NDS but are rarely used in residential wood construction. The applicable sections of the NDS related to connection design as covered in this chapter include

- NDS•7–Mechanical Connections (General Requirements);
- NDS•8–Bolts;
- NDS•9–Lag Screws; and
- NDS•12–Nails and Spikes.

While wood connections are generally responsible for the complex, non-linear behavior of wood structural systems, the design procedures outlined in the NDS are straightforward. The NDS connection values are generally conservative from a structural safety standpoint. Further, the NDS's basic or tabulated design values are associated with tests of single fasteners in standardized conditions. As a result, the NDS provides several adjustments to account for various factors that alter the performance of a connection; in particular, the performance of wood connections is highly dependent on the species (i.e., density or specific gravity) of wood. Table 7.3 provides the specific gravity values of various wood species typically used in house construction.

TABLE 7.3 Common Framing Lumber Species and Specific Gravity Values

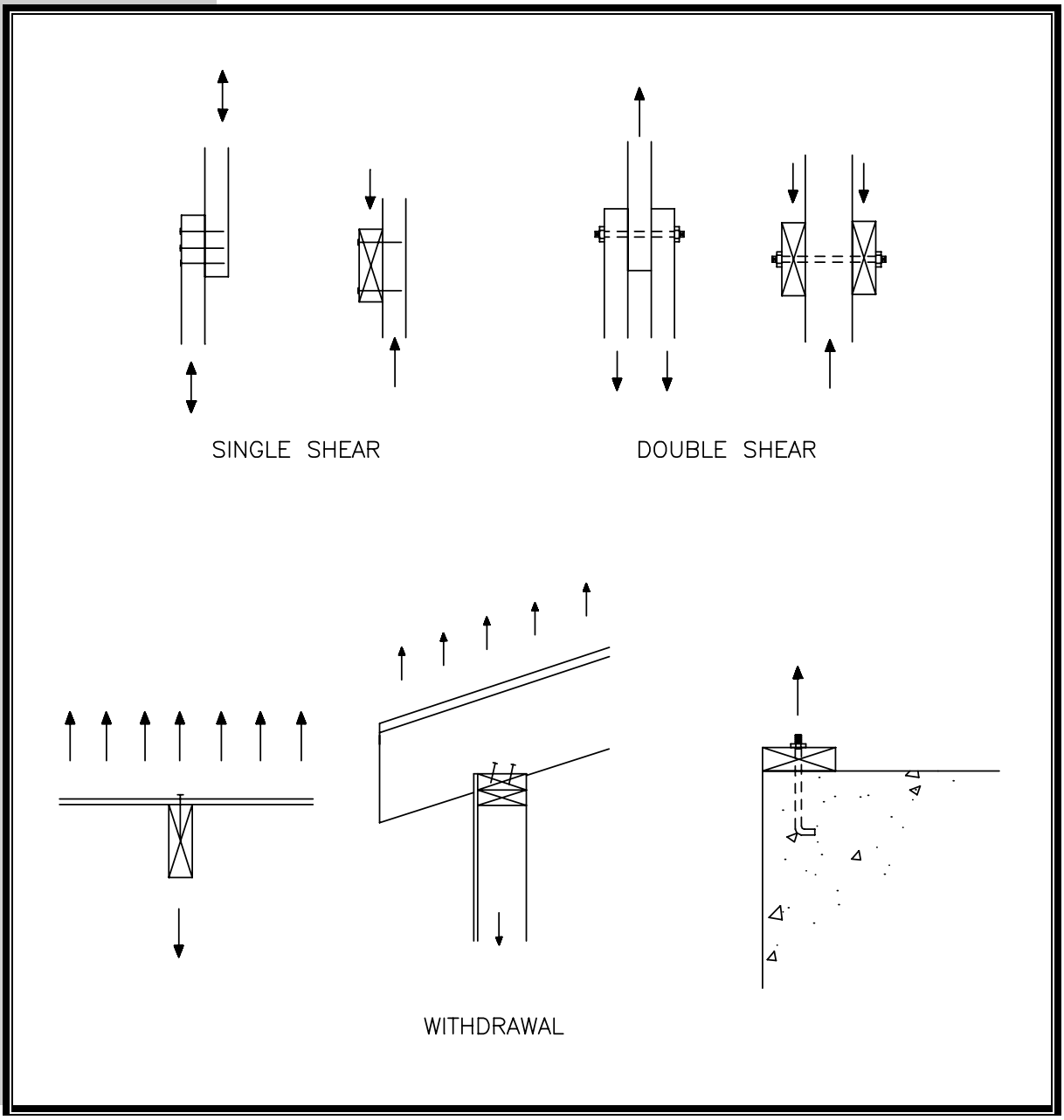
Lumber Species	Specific Gravity, G
Southern Pine (SP)	0.55
Douglas Fir-Larch (DF-L)	0.50
Hem-Fir (HF)	0.43
Spruce-Pine-Fir (SPF)	0.42
Spruce-Pine-Fir (South)	0.36

The moisture condition of the wood is also critical to long-term connection performance, particularly for nails in withdrawal. In some cases, the withdrawal value of fasteners installed in moist lumber can decrease by as much as 50 percent over time as the lumber dries to its equilibrium moisture content. At the same time, a nail may develop a layer of rust that increases withdrawal capacity. In contrast, deformed shank nails tend to hold their withdrawal capacity much more reliably under varying moisture and use conditions. For this and other reasons, the design nail withdrawal capacities in the NDS for smooth shank nails are based on a fairly conservative reduction factor, resulting in about one-fifth of the average ultimate tested withdrawal capacity. The reduction includes a safety factor as well as a load duration adjustment (i.e., decreased by a factor of 1.6 to adjust from short-term tests to normal duration load). Design values for nails and bolts in shear are based on a deformation (i.e., slip) limit state and not their ultimate capacity, resulting in a safety factor that may range from 3 to 5 based on ultimate tested capacities. One argument for retaining a high safety factor in shear connections is that the joint may creep under long-term load. While creep is not a concern for many joints, slip of joints in a trussed assembly (i.e., rafter-ceiling joist roof framing) is critical and, in key joints, can result in a magnified deflection of the assembly over time (i.e., creep).

In view of the above discussion, there are a number of uncertainties in the design of connections that can lead to conservative or unconservative designs relative to the intent of the NDS and practical experience. The designer is advised to follow the NDS procedures carefully, but should be prepared to make practical adjustments as dictated by sound judgment and experience and allowed in the NDS; refer to NDS•7.1.1.4.

Withdrawal design values for nails and lag screws in the NDS are based on the fastener being oriented perpendicular to the grain of the wood. Shear design values in wood connections are also based on the fastener being oriented perpendicular to the grain of wood. However, the lateral (shear) design values are dependent on the direction of loading relative to the wood grain direction in each of the connected members. Refer to Figure 7.4 for an illustration of various connection types and loading conditions.

FIGURE 7.4 Types of Connections and Loading Conditions



The NDS provides tabulated connection design values that use the following symbols for the three basic types of loading:

- W–withdrawal (or tension loading);
- $Z_{\perp}$–shear perpendicular to wood grain; and
- $Z_{\parallel}$–shear parallel to wood grain.

In addition to the already tabulated design values for the above structural resistance properties of connections, the NDS provides calculation methods to address conditions that may not be covered by the tables and that give more

flexibility to the design of connections. The methods are appropriate for use in hand calculations or with computer spreadsheets.

For withdrawal, the design equations are relatively simple empirical relationships (based on test data) that explain the effect of fastener size (diameter), penetration into the wood, and density of the wood. For shear, the equations are somewhat more complex because of the multiple failure modes that may result from fastener characteristics, wood density, and size of the wood members. Six shear-yielding modes (and a design equation for each) address various yielding conditions in either the wood members or the fasteners that join the members. The critical yield mode is used to determine the design shear value for the connection. Refer to NDS•Appendix I for a description of the yield modes.

The yield equations in the NDS are based on general dowel equations that use principles of engineering mechanics to predict the shear capacity of a doweled joint. The general dowel equations can be used with joints that have a gap between the members and they can also be used to predict ultimate capacity of a joint made of wood, wood and metal, or wood and concrete. However, the equations do not account for friction between members or the anchoring/cinching effect of the fastener head as the joint deforms and the fastener rotates or develops tensile forces. These effects are important to the ultimate capacity of wood connections in shear and, therefore, the general dowel equations may be considered to be conservative; refer to Section 7.3.6. For additional guidance and background on the use of the general dowel equations, refer to the NDS *Commentary* and other useful design resources available through the American Forest & Paper Association (AF&PA, 1999; Showalter, Line, and Douglas, 1999).

7.3.2 Adjusted Allowable Design Values

Design values for wood connections are subject to adjustments in a manner similar to that required for wood members themselves (see Section 5.2.4 of Chapter 5). The calculated or tabulated design values for W and Z are multiplied by the applicable adjustment factors to determine adjusted allowable design values, Z' and W', as shown below for the various connection methods (i.e., nails, bolts, and lag screws).

[NDS•12.3 & 7.3]

$$Z' = ZC_D C_M C_t C_g C_\Delta \quad \text{for bolts}$$

$$Z' = ZC_D C_M C_t C_g C_\Delta C_d C_{eg} \quad \text{for lag screws}$$

$$Z' = ZC_D C_M C_t C_d C_{eg} C_{di} C_{tn} \quad \text{for nails and spikes}$$

[NDS•12.2&7.3]

$$W' = WC_D C_M C_t C_{tn} \quad \text{for nails and spikes}$$

$$W' = WC_D C_M C_t C_{eg} \quad \text{for lag screws}$$

The adjustment factors and their applicability to wood connection design are briefly described as follows:

- C_D–*Load Duration Factor* (NDS•2.3.2 and Chapter 5, Table 5.3)–applies to W and Z values for all fasteners based on design load duration but shall not exceed 1.6 (i.e., wind and earthquake load duration factor).
- C_M–*Wet Service Factor* (NDS•7.3.3)–applies to W and Z values for all connections based on moisture conditions at the time of fabrication and during service; not applicable to residential framing.
- C_t–*Temperature Factor* (NDS•7.3.4)–applies to the W and Z values for all connections exposed to sustained temperatures of greater than 100^oF; not typically used in residential framing.
- C_g–*Group Action Factor* (NDS•7.3.6)–applies to Z values of two or more bolts or lag screws loaded in single or multiple shear and aligned in the direction of the load (i.e., rows).
- C_Δ–*Geometry Factor* (NDS•8.5.2, 9.4.)–applies to the Z values for bolts and lag screws when the end distance or spacing of the bolts is less than assumed in the unadjusted design values.
- C_d–*Penetration Depth Factor* (NDS•9.3.3, 12.3.4)–applies to the Z values of lag screws and nails when the penetration into the main member is less than 8D for lag screws or 12D for nails (where D = shank diameter); sometimes applicable to residential nailed connections.
- C_{eg}–*End Grain Factor* (NDS•9.2.2, 9.3.4, 12.3.5)–applies to W and Z values for lag screws and to Z values for nails to account for reduced capacity when the fastener is inserted into the end grain (C_{eg}=0.67).
- C_{di}–*Diaphragm Factor* (NDS•12.3.6)–applies to the Z values of nails only to account for system effects from multiple nails used in sheathed diaphragm construction (C_{di} = 1.1).
- C_{tn}–*Toenail Factor* (NDS•12.3.7)–applies to the W and Z values of toenailed connections (C_{tn} = 0.67 for withdrawal and = 0.83 for shear). It does not apply to slant nailing in withdrawal or shear; refer to Section 7.3.6.

The total allowable design value for a connection (as adjusted by the appropriate factors above) must meet or exceed the design load determined for the connection (refer to Chapter 3 for design loads). The values for W and Z are based on single fastener connections. In instances of connections involving multiple fasteners, the values for the individual or single fastener can be summed to determine the total connection design value only when C_g is applied (to bolts and lag screws only) and fasteners are the same type and similar size. However, this approach may overlook certain system effects that can improve the actual

performance of the joint in a constructed system or assembly (see Section 7.3.6). Conditions that may decrease estimated performance, such as prying action induced by the joint configuration and/or eccentric loads and other factors should also be considered.

In addition, the NDS does not provide values for nail withdrawal or shear when wood structural panel members (i.e., plywood or oriented strand board) are used as a part of the joint. This type of joint–wood member to structural wood panel–occurs frequently in residential construction. Z values can be estimated by using the yield equations for nails in NDS 12.3.1 and assuming a reasonable specific gravity (density) value for the wood structural panels, such as G = 0.5. W values for nails in wood structural panels can be estimated in a similar fashion by using the withdrawal equation presented in the next section. The tabulated W and Z values in NDS•12 may also be used, but with some caution as to the selected table parameters.

7.3.3 Nailed Connections

The procedures in NDS•12 provide for the design of nailed connections to resist shear and withdrawal loads in wood-to-wood and metal-to-wood connections. As mentioned, many specialty "nail-type" fasteners are available for wood-to-concrete and even wood-to-steel connections. The designer should consult manufacturer data for connection designs that use proprietary fastening systems.

The withdrawal strength of a smooth nail (driven into the side grain of lumber) is determined in accordance with either the empirical design equation below or NDS•Table 12.2A.

[NDS•12.2.1]

$$W = 1380(G)^{\frac{5}{2}} DL_p$$ unadjusted withdrawal design value (lb) for a smooth shank nail

where,

G = specific gravity of the lumber member receiving the nail tip
D = the diameter of the nail shank (in)
L_p = the depth of penetration (in) of the nail into the member receiving the nail tip

The design strength of nails is greater when a nail is driven into the side rather than the end grain of a member. Withdrawal information is available for nails driven into the side grain; however, the withdrawal capacity of a nail driven into the end grain is assumed to be zero because of its unreliability. Furthermore, the NDS does not provide a method for determining withdrawal values for deformed shank nails. These nails significantly enhance withdrawal capacity and are frequently used to attach roof sheathing in high-wind areas. They are also used to attach floor sheathing and some siding materials to prevent nail "back-out." The use of deformed shank nails is usually based on experience or preference.

The design shear value, Z, for a nail is typically determined by using the following tables from NDS•12:

- Tables 12.3A and B. Nailed wood-to-wood, single-shear (two-member) connections with the same species of lumber using box or common nails, respectively.
- Tables 12.3E and F. Nailed metal plate-to-wood connections using box or common nails, respectively.

The yield equations in NDS•12.3 may be used for conditions not represented in the design value tables for Z. Regardless of the method used to determine the Z value for a single nail, the value must be adjusted as described in Section 7.3.2. As noted in the NDS, the single nail value is used to determine the design value.

It is also worth mentioning that the NDS provides an equation for determining allowable design value for shear when a nailed connection is loaded in combined withdrawal and shear (see NDS•12.3.8, Equation 12.3-6). The equation appears to be most applicable to a gable-end truss connection to the roof sheathing under conditions of roof sheathing uplift and wall lateral load owing to wind. The designer might contemplate other applications but should take care in considering the combination of loads that would be necessary to create simultaneous uplift and shear worthy of a special calculation.

7.3.4 Bolted Connections

Bolts may be designed in accordance with NDS•8 to resist shear loads in wood-to-wood, wood-to-metal, and wood-to-concrete connections. As mentioned, many specialty “bolt-type” fasteners can be used to connect wood to other materials, particularly concrete and masonry. One common example is an epoxy-set anchor. Manufacturer data should be consulted for connection designs that use proprietary fastening systems.

The design shear value Z for a bolted connection is typically determined by using the following tables from NDS•8:

- Table 8.2A. Bolted wood-to-wood, single-shear (two-member) connections with the same species of lumber.
- Table 8.2B. Bolted metal plate-to-wood, single-shear (two-member) connections; metal plate thickness of 1/4-inch minimum.
- Table 8.2D. Bolted single-shear wood-to-concrete connections; based on minimum 6-inch bolt embedment in minimum f_c = 2,000 psi concrete.

The yield equations of NDS•8.2 (single-shear joints) and NDS•8.3 (double-shear joints) may be used for conditions not represented in the design value tables. Regardless of the method used to determine the Z value for a single bolted connection, the value must be adjusted as described in Section 7.3.2.

It should be noted that the NDS does not provide W values for bolts. The tension value of a bolt connection in wood framing is usually limited by the bearing capacity of the wood as determined by the surface area of a washer used underneath the bolt head or nut. When calculating the bearing capacity of the wood based on the tension in a bolted joint, the designer should use the small

bearing area value C_b to adjust the allowable compressive stress perpendicular to grain $F_{c\perp}$ (see NDS•2.3.10). It should also be remembered that the allowable compressive stress of lumber is based on a deformation limit state, not capacity; refer to Section 5.2.3 of Chapter 5. In addition, the designer should verify the tension capacity of the bolt and its connection to other materials (i.e., concrete or masonry as covered in Section 7.4). The bending capacity of the washer should also be considered. For example, a wide but thin washer will not evenly distribute the bearing force to the surrounding wood.

The arrangement of bolts and drilling of holes are extremely important to the performance of a bolted connection. The designer should carefully follow the minimum edge, end, and spacing requirements of NDS•8.5. When necessary, the designer should adjust the design value for the bolts in a connection by using the geometry factor C_ρ and the group action factor C_g discussed in Section 7.3.2.

Any possible torsional load on a bolted connection (or any connection for that manner) should also be considered in accordance with the NDS. In such conditions, the pattern of the fasteners in the connection can become critical to performance in resisting both a direct shear load and the loads created by a torsional moment on the connection. Fortunately, this condition is not often applicable to typical light-frame construction. However, cantilevered members that rely on connections to "anchor" the cantilevered member to other members will experience this effect, and the fasteners closest to the cantilever span will experience greater shear load. One example of this condition sometimes occurs with balcony construction in residential buildings; failure to consider the effect discussed above has been associated with some notable balcony collapses.

For wood members bolted to concrete, the design lateral values are provided in NDS•Table8.2E. The yield equations (or general dowel equations) may also be used to conservatively determine the joint capacity. A recent study has made recommendations regarding reasonable assumptions that must be made in applying the yield equations to bolted wood-to-concrete connections (Stieda, et al., 1999). Using symbols defined in the NDS, the study recommends an R_e value of 5 and an R_t value of 3. These assumptions are reported as being conservative because fastener head effects and joint friction are ignored in the general dowel equations.

7.3.5 Lag Screws

Lag screws (or lag bolts) may be designed to resist shear and withdrawal loads in wood-to-wood and metal-to-wood connections in accordance with NDS•9. As mentioned, many specialty "screw-type" fasteners can be installed in wood. Some tap their own holes and do not require predrilling. Manufacturer data should be consulted for connection designs that use proprietary fastening systems.

The withdrawal strength of a lag screw (inserted into the side grain of lumber) is determined in accordance with either the empirical design equation below or NDS•Table 9.2A. It should be noted that the equation below is based on single lag screw connection tests and is associated with a reduction factor of 0.2 applied to average ultimate withdrawal capacity to adjust for load duration and safety. Also, the penetration length of the lag screw L_p into the main member does

not include the tapered portion at the point. NDS•Appendix L contains dimensions for lag screws.

[NDS•9.2.1]

$$W = 1800(G)^{\frac{3}{2}} D^{\frac{3}{4}} L_p$$ unadjusted withdrawal design value (lb) for a lag screw

where,

G = specific gravity of the lumber receiving the lag screw tip
D = the diameter of the lag screw shank (in)
L_p = the depth of penetration (in) of the lag screw into the member receiving the tip, less the tapered length of the tip

The allowable withdrawal design strength of a lag screw is greater when the screw is installed in the side rather than the end grain of a member. However, unlike the treatment of nails, the withdrawal strength of lag screws installed in the end grain may be calculated by using the C_{eg} adjustment factor with the equation above.

The design shear value Z for a lag screw is typically determined by using the following tables from NDS•9:

- Table 9.3A. Lag screw, single-shear (two-member) connections with the same species of lumber for both members.
- Table 9.3B. Lag screw and metal plate-to-wood connections.

The yield equations in NDS•9.3 may be used for conditions not represented in the design value tables for Z. Regardless of the method used to determine the Z value for a single lag screw, the value must be adjusted as described in Section 7.3.2.

It is also worth mentioning that the NDS provides an equation for determining the allowable shear design value when a lag screw connection is loaded in combined withdrawal and shear (see NDS•9.3.5, Equation 9.3-6). The equation does not, however, appear to apply to typical uses of lag screws in residential construction.

7.3.6 System Design Considerations

As with any building code or design specification, the NDS provisions may or may not address various conditions encountered in the field. Earlier chapters made several recommendations regarding alternative or improved design approaches. Similarly, some considerations regarding wood connection design are in order.

First, as a general design consideration, “crowded” connections should be avoided. If too many fasteners are used (particularly nails), they may cause splitting during installation. When connections become “crowded,” an alternative fastener or connection detail should be considered. Basically, the connection detail should be practical and efficient.

Second, while the NDS addresses “system effects” within a particular joint (i.e., element) that uses multiple bolts or lag screws (i.e. the group action factor C_g), it does not include provisions regarding the system effects of multiple joints

in an assembly or system of components. Therefore, some consideration of system effects is given below based on several relevant studies related to key connections in a home that allow the dwelling to perform effectively as a structural unit.

Sheathing Withdrawal Connections

Several recent studies have focused on roof sheathing attachment and nail withdrawal, primarily as a result of Hurricane Andrew (HUD, 1999a; McClain, 1997; Cunningham, 1993; Mizzell and Schiff, 1994; and Murphy, Pye, and Rosowsky, 1995); refer to Chapter 1. The studies identify problems related to predicting the pull-off capacity of sheathing based on single nail withdrawal values and determining the tributary withdrawal load (i.e., wind suction pressure) on a particular sheathing fastener. One clear finding, however, is that the nails on the interior of the roof sheathing panels are the critical fasteners (i.e., initiate panel withdrawal failure) because of the generally larger tributary area served by these fasteners. The studies also identified benefits to the use of screws and deformed shank nails. However, use of a standard geometric tributary area of the sheathing fastener and the wind loads in Chapter 3, along with the NDS withdrawal values (Section 7.3.3), will generally result in a reasonable design using nails. The wind load duration factor should also be applied to adjust the withdrawal values since a commensurate reduction is implicit in the design withdrawal values relative to the short-term, tested, ultimate withdrawal capacities (see Section 7.3).

It is interesting, however, that one study found that the lower-bound (i.e., 5th percentile) sheathing pull-off resistance was considerably higher than that predicted by use of single-nail test values (Murphy, Pye, and Rosowsky, 1995). The difference was as large as a factor of 1.39 greater than the single nail values. While this would suggest a withdrawal system factor of at least 1.3 for sheathing nails, it should be subject to additional considerations. For example, sheathing nails are placed by people using tools in somewhat adverse conditions (i.e., on a roof), not in a laboratory. Therefore, this system effect may be best considered as a reasonable "construction tolerance" on actual nail spacing variation relative to that intended by design. Thus, an 8- to 9-inch nail spacing on roof sheathing nails in the panel's field could be "tolerated" when a 6-inch spacing is "targeted" by design.

Roof-to-Wall Connections

A couple of studies (Reed, et al., 1996; Conner, et al., 1987) have investigated the capacity of roof-to-wall (i.e., sloped rafter to top plate) connections using conventional toenailing and other enhancements (i.e., strapping, brackets, gluing, etc.). Again, the primary concern is related to high wind conditions, such as experienced during Hurricane Andrew and other extreme wind events; refer to Chapter 1.

First, as a matter of clarification, the toenail reduction factor C_{tn} does not apply to *slant-nailing* such as those used for rafter-to-wall connections and floor-to-wall connections in conventional residential construction (Hoyle and Woeste, 1989). Toenailing occurs when a nail is driven at an angle in a direction parallel-

to-grain at the end of a member (i.e., a wall stud toenail connection to the top or bottom plate that may be used instead of end nailing). Slant nailing occurs when a nail is driven at an angle, but in a direction perpendicular-to-grain through the side of the member and into the face grain of the other (i.e., from a roof rafter or floor band joist to a wall top plate). Though a generally reliable connection in most homes and similar structures built in the United States, even a well-designed slant-nail connection used to attach roofs to walls will become impractical in hurricane-prone regions or similar high-wind areas. In these conditions, a metal strap or bracket is preferrable.

Based on the studies of roof-to-wall connections, five key findings are summarized as follows (Reed et al., 1996; Conner et al., 1987):

1. In general, it was found that slant-nails (not to be confused with toe-nails) in combination with metal straps or brackets do not provide directly additive uplift resistance.
2. A basic metal twist strap placed on the interior side of the walls (i.e., gypsum board side) resulted in top plate tear-out and premature failure. However, a strap placed on the outside of the wall (i.e., structural sheathing side) was able to develop its full capacity without additional enhancement of the conventional stud-to-top plate connection (see Table 7.1).
3. The withdrawal capacity for single joints with slant nails was reasonably predicted by NDS with a safety factor of about 2 to 3.5. However, with multiple joints tested simultaneously, a system factor on withdrawal capacity of greater than 1.3 was found for the slant-nailed rafter-to-wall connection. A similar system effect was not found on strap connections, although the strap capacity was substantially higher. The ultimate capacity of the simple strap connection (using five 8d nails on either side of the strap–five in the spruce rafter and five in the southern yellow pine top plate) was found to be about 1,900 pounds per connection. The capacity of three 8d common slant nails used in the same joint configuration was found to be 420 pounds on average, and with higher variation. When the three 8d common toenail connection was tested in an assembly of eight such joints, the average ultimate withdrawal capacity per joint was found to be 670 pounds with a somewhat lower variation. Similar “system” increases were not found for the strap connection. The 670 pounds capacity was similar to that realized for a rafter-to-wall joint using three 16d box nails in Douglas fir framing.
4. It was found that the strap manufacturer’s published value had an excessive safety margin of greater than 5 relative to average ultimate capacity. Adjusted to an appropriate safety factor in the range of 2 to 3 (as calculated by applying NDS nail shear equations by using a metal side plate), the strap (a simple 18g twist strap) would cover a multitude of high wind conditions with a simple, economical connection detail.
5. The use of deformed shank (i.e., annular ring) nails was found to increase dramatically the uplift capacity of the roof-to-wall connections using the slant nailing method.

Heel Joint in Rafter-to-Ceiling Joist Connections

The heel joint connection at the intersection of rafters and ceiling joists have long been considered one of the weaker connections in conventional wood roof framing. In fact, this highly stressed joint is one of the accolades of using a wood truss rather than conventional rafter framing (particularly in high-wind or snow-load conditions). However, the performance of conventional rafter-ceiling joist heel joint connections should be understood by the designer since they are frequently encountered in residential construction.

First, conventional rafter and ceiling joist (cross-tie) framing is simply a "site-built" truss. Therefore, the joint loads can be analyzed by using methods that are applicable to trusses (i.e., pinned joint analysis). However, the performance of the system should be considered. As mentioned earlier for roof trusses (Section 5.6.1 in Chapter 5), a system factor of 1.1 is applicable to tension members and connections. Therefore, the calculated shear capacity of the nails in the heel joint (and in ceiling joist splices) may be multiplied by a system factor of 1.1, which is considered conservative. Second, it must be remembered that the nail shear values are based on a deformation limit and generally have a conservative safety factor of three to five relative to the ultimate capacity. Finally, the nail values should be adjusted for duration of load (i.e., snow load duration factor of 1.15 to 1.25); refer to Section 5.2.4 of Chapter 5. With these considerations and with the use of rafter support braces at or near mid-span (as is common), reasonable heel joint designs should be possible for most typical design conditions in residential construction.

Wall-to-Floor Connections

When wood sole plates are connected to wood floors, many nails are often used, particularly along the total length of the sole plate or wall bottom plate. When connected to a concrete slab or foundation wall, there are usually several bolts along the length of the bottom plate. This points toward the question of possible system effects in estimating the shear capacity (and uplift capacity) of these connections for design purposes.

In recent shear wall tests, walls connected with pneumatic nails (0.131-inch diameter by 3 inches long) spaced in pairs at 16 inches on center along the bottom plate were found to resist over 600 pounds in shear per nail (HUD, 1999b). The bottom plate was Spruce-Pine-Fir lumber and the base beam was Southern Yellow Pine. This value is about 4.5 times the adjusted allowable design shear capacity predicted by use of the NDS equations. Similarly, connections using 5/8-inch-diameter anchor bolts at 6 feet on center (all other conditions equal) were tested in full shear wall assemblies; the ultimate shear capacity per bolt was found to be 4,400 pounds. This value is about 3.5 times the adjusted allowable design shear capacity per the NDS equations. These safety margins appear excessive and should be considered by the designer when evaluating similar connections from a practical "system" standpoint.

7.4 Design of Concrete and Masonry Connections

7.4.1 General

In typical residential construction, the interconnection of concrete and masonry elements or systems is generally related to the foundation and usually handled in accordance with standard or accepted practice. The bolted wood member connections to concrete as in Section 7.3.4 are suitable for bolted wood connections to properly grouted masonry (see Chapter 4). Moreover, numerous specialty fasteners or connectors (including power driven and cast-in-place) can be used to fasten wood materials to masonry or concrete. The designer should consult the manufacturer's literature for available connectors, fasteners, and design values.

This section discusses some typical concrete and masonry connection designs in accordance with the ACI 318 concrete design specification and ACI 530 masonry design specification (ACI, 1999a; ACI, 1999b).

7.4.2 Concrete or Masonry Foundation Wall to Footing

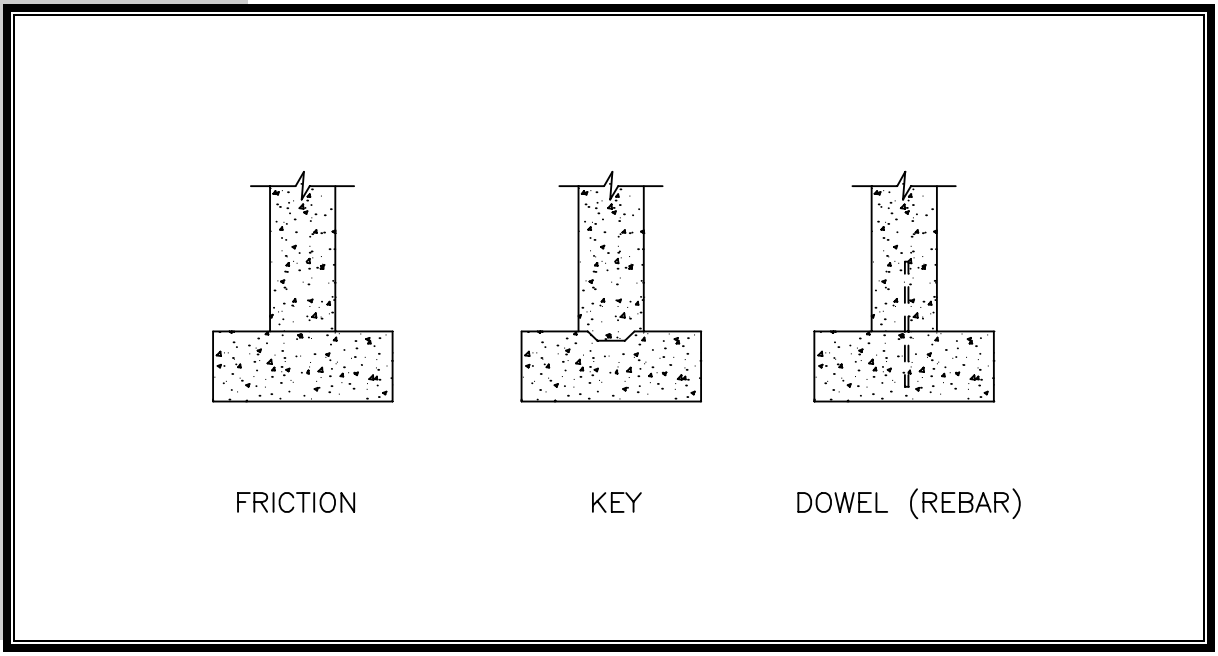
Footing connections, if any, are intended to transfer shear loads from the wall to the footing below. The shear loads are generally produced by lateral soil pressure acting on the foundation (see Chapter 3).

Footing-to-wall connections for residential construction are constructed in any one of the following three ways (refer to Figure 7.5 for illustrations of the connections):

- no vertical reinforcement or key;
- key only; or
- dowel only.

Generally, no special connection is needed in nonhurricane-prone or low- to moderate-hazard seismic areas. Instead, friction is sufficient for low, unbalanced backfill heights while the basement slab can resist slippage for higher backfill heights on basement walls. The basement slab abuts the basement wall near its base and thus provides lateral support. If gravel footings are used, the unbalanced backfill height needs to be sufficiently low (i.e., less than 3 feet), or means must be provided to prevent the foundation wall from slipping sideways from lateral soil loads. Again, a basement slab can provide the needed support. Alternatively, a footing key or doweled connection can be used.

FIGURE 7.5 ***Concrete or Masonry Wall-to-Footing Connections***



Friction Used to Provide Shear Transfer

To verify the amount of shear resistance provided by friction alone, assume a coefficient of friction between two concrete surfaces of $\mu = 0.6$. Using dead loads only, determine the static friction force, $F = \mu NA$, where *F* is the friction force (lb), *N* is the dead load (psf), and *A* is the bearing surface area (sf) between the wall and the footing.

Key Used to Provide Shear Transfer

A concrete key is commonly used to "interlock" foundation walls to footings. If foundation walls are constructed of masonry, the first course of masonry must be grouted solid when a key is used.

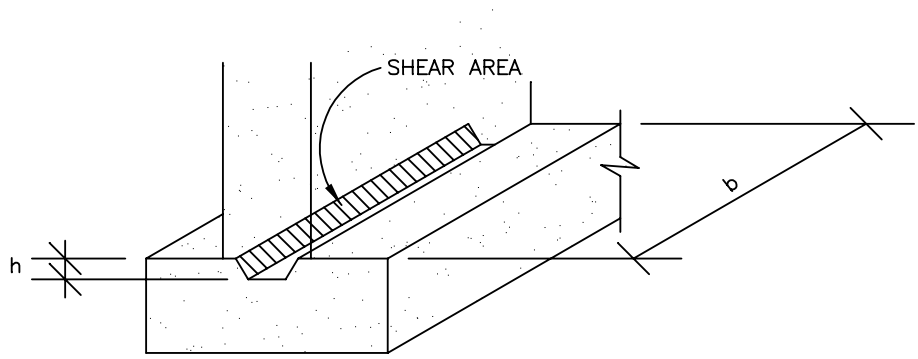
In residential construction, a key is often formed by using a 2x4 wood board with chamfered edges that is placed into the surface of the footing immediately after the concrete pour. Figure 7.6 illustrates a footing with a key. Shear resistance developed by the key is computed in accordance with the equation below.

[ACI-318•22.5]

$$V_u \le \phi V_n$$

$$V_n = \frac{4}{3}\sqrt{f'_c}\,bh$$

FIGURE 7.6 Key in Concrete Footings



Dowels Used to Provide Adequate Shear Transfer

Shear forces at the base of exterior foundation walls may require a dowel to transfer the forces from the wall to the footing. The equations below described by ACI-318 as the Shear-Friction Method are used to develop shear resistance with vertical reinforcement (dowels) across the wall-footing interface.

[ACI-318•11.7]

Masonry Walls

$$l_{be} \geq 12 d_b$$

$$B_v = \text{minimum of} \begin{Bmatrix} 350 \sqrt[4]{f'_m A_v} \\ 0.12 A_v f_y \end{Bmatrix}$$

Concrete Walls

$$V_u \leq \phi V_n$$

$$V_n = A_{vf} f_y \mu \leq \begin{Bmatrix} 0.2 f'_c A_c \\ 800 A_c \end{Bmatrix}$$

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

$$\phi = 0.85$$

If dowels are used to transfer shear forces from the base of the wall to the footing, use the equations below to determine the minimum development length required (refer to Figure 7.7 for typical dowel placement). If development length exceeds the footing thickness, the dowel must be in the form of a hook, which is rarely required in residential construction.

[ACI-318•12.2, 12.5] Concrete Walls

Standard Hooks

$$l_{hb} = \frac{1200 d_b}{\sqrt{f'_c}} \quad \text{where } f_y = 60{,}000 \text{ psi}$$

Deformed Bars

$$l_{db} = \left(\frac{3 f_y}{40\sqrt{f'_c}} \right) \left(\frac{\alpha\beta\gamma\lambda}{\frac{c + K_{TR}}{d_b}} \right)$$

$$\frac{c + K_{TR}}{d_b} \le 2.5$$

$$\xi = \frac{f_y}{60{,}000}$$

$$l_d = l_{db} \left(\frac{A_{s,required}}{A_{s,provided}} \right) \ge 12''$$

$$\omega = \frac{A_{s,required}}{A_{s,provided}}$$

[ACI-530•1.12.3,2.1.8] Masonry Walls

Standard Hooks

$$l_d = 0.0015 d_b F_s \ge 12 \text{ in.}$$

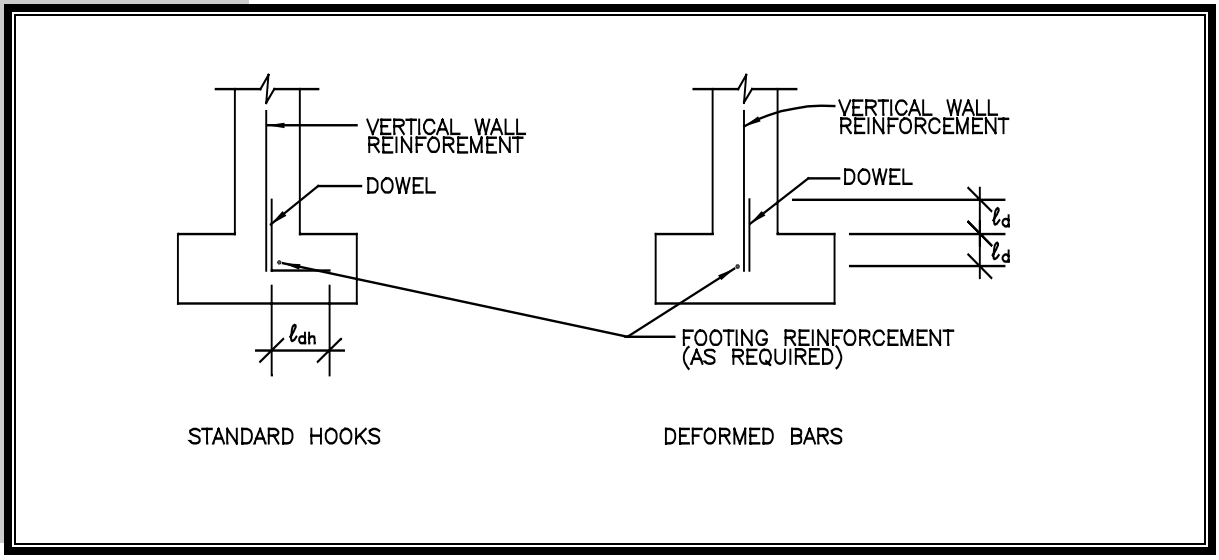
$$l_e = 11.25 d_b$$

Deformed Bars

$$l_d = \text{maximum } \{12 d_b\}$$

$$l_d = \text{maximum } \{12 d_b\}$$

FIGURE 7.7 Dowel Placement in Concrete Footings



The minimum embedment length is a limit specified in ACI-318 that is not necessarily compatible with residential construction conditions and practice. Therefore, this guide suggests a minimum embedment length of 6 to 8 inches for footing dowels, when necessary, in residential construction applications. In addition, dowels are sometimes used in residential construction to connect other concrete elements, such as porch slabs or stairs, to the house foundation to control differential movement. However, exterior concrete “flat work” adjacent to a home should be founded on adequate soil bearing or reasonably compacted backfill. Finally, connecting exterior concrete work to the house foundation requires caution, particularly in colder climates and soil conditions where frost heave may be a concern.

7.4.3 Anchorage and Bearing on Foundation Walls

Anchorage Tension (Uplift) Capacity

The equations below determine whether the concrete or masonry shear area of each bolt is sufficient to resist pull-out from the wall as a result of uplift forces and shear friction in the concrete.

[ACI-318•11.3, ACI-530•2.1.2]

Concrete Foundation Wall

$$V_u \le \phi V_c$$

$$V_c = 4A_v\sqrt{f'_c}$$

$$A_v = \text{minimum of} \begin{Bmatrix} \pi l_b^{\,2} \\ \pi h^2 \end{Bmatrix}$$

Masonry Foundation Wall

$$b_a \le B_a$$

$$B_a = \text{minimum of} \begin{Bmatrix} 0.5A_p\sqrt{f'_m} \\ 0.2A_b f_y \end{Bmatrix}$$

$$A_p = \text{minimum of} \begin{Bmatrix} \pi l_b^{\,2} \\ \pi l_{be}^{\,2} \end{Bmatrix}$$

Bearing Strength

Determining the adequacy of the bearing strength of a foundation wall follows ACI-318•10.17 for concrete or ACI-530•2.1.7 for masonry. The bearing strength of the foundation wall is typically adequate for the loads encountered in residential construction.

[ACI-318•10.17 and ACI-530•2.1.7]

Concrete Foundation Wall

B_c = factored bearing load

$$B_c \le \phi 0.85 f'_c A_1$$

$$\phi = 0.7$$

Masonry Foundation Wall

$$f_a \le F_a$$

$$f_a = \frac{P}{A_1}$$

$$F_a \le 0.25 f'_m$$

When the foundation wall's supporting surface is wider on all sides than the loaded area, the designer is permitted to determine the design bearing strength on the loaded area by using the equations below.

[ACI-318•10.7 and ACI-530•2.1.7]

Concrete Foundation Wall

$$B_c = \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \quad \text{where} \quad \sqrt{\frac{A_2}{A_1}} \le 2$$

Masonry Foundation Wall

$$f_a = \frac{P}{A_1\sqrt{A_2/A_1}} \quad \text{where} \quad \sqrt{\frac{A_2}{A_1}} \le 2$$

7.5 Design Examples

EXAMPLE 7.1 Roof Sheathing Connections

Given

- Design wind speed is 130 mph gust with an open (coastal) exposure
- Two-story home with a gable roof
- Roof framing lumber is Southern Yellow Pine (G=0.55)
- Roof framing is spaced at 24 inches on center
- Roof sheathing is 7/16-inch-thick structural wood panel

Find

1. Wind load (suction) on roof sheathing.
2. Nail type/size and maximum spacing.

Solution

1. Determine the wind load on roof sheathing (Chapter 3, Section 3.6.2)

Step 1: Basic velocity pressure	=	24.6 psf	(Table 3.7)
Step 2: Adjust for open exposure	=	1.4(24.6 psf) = 34.4 psf	
Step 3: Skip			
Step 4: Roof sheathing G_{cp}	=	-2.2	(Table 3.9)
Step 5: Design load	=	(-2.2)(34.4 psf) = 76 psf	

2. Select a trial nail type and size, determine withdrawal capacity, and calculated required spacing

Use an 8d pneumatic nail (0.113 inch diameter) with a length of 2 3/8 inches. The unadjusted design withdrawal capacity is determined using the equation in Section 7.3.3.

$$W = 1380(G)^{2.5}DL_p$$

$G = 0.55$
$D = 0.113$ in
$L_p = (2\ 3/8\text{ in}) - (7/16\text{ in}) = 1.9$ in

$$W = 1380(0.55)^{2.5}(0.113\text{ in})(1.9\text{ in}) = 66.5\text{ lb}$$

Determine the adjusted design withdrawal capacity using the applicable adjustment factors discussed in Section 7.3.2.

$$W' = WC_D = (66.5\text{ lb})(1.6) = 106\text{ lb}$$

Determine the required nail spacing in the roof sheathing panel interior.

Tributary sheathing area = (roof framing spacing)(nail spacing)
= (2 ft)(s)
Withdrawal load per nail = (wind uplift pressure)(2 ft)(s)
= (76 psf)(2 ft)(s)

W' ≥ design withdrawal load
106 lb ≥ (76 psf)(2 ft)(s)
s ≤ 0.69 ft

Use a maximum nail spacing of 8 inches in the roof sheathing panel interior.

Notes:

1. If Spruce-Pine-Fir (G=0.42) roof framing lumber is substituted, W' would be 54 lb and the required nail spacing would reduce to 4 inches on center in the roof sheathing panel interior. Thus, it is extremely important to carefully consider and verify the species of framing lumber when determining fastening requirements for roof sheathing.
2. The above analysis is based on a smooth shank nail. A ring shank nail may be used to provide greater withdrawal capacity that is also less susceptible to lumber moisture conditions at installation and related long-term effects on withdrawal capacity.
3. With the smaller tributary area, the roof sheathing edges that are supported on framing members may be fastened at the standard 6 inch on center fastener spacing. For simplicity, it may be easier to specify a 6 inch on center spacing for all roof sheathing fasteners, but give an allowance of 2 to 3 inches for a reasonable construction tolerance; refer to Section 7.3.6.
4. As an added measure given the extreme wind environment, the sheathing nail spacing along the gable end truss/framing should be specified at a closer spacing, say 4 inches on center. These fasteners are critical to the performance of light-frame gable roofs in extreme wind events; refer to the discussion on hurricanes in Chapter 1. NDS•12.3.8 provides an equation to determine nail lateral strength when subjected to a combined lateral and withdrawal load. This equation may be used to verify the 4 inch nail spacing recommendation at the gable end.

Conclusion

This example problem demonstrates a simple application of the nail withdrawal equation in the NDS. The withdrawal forces on connections in residential construction are usually of greatest concern in the roof sheathing attachment. In hurricane prone regions, it is common practice to use a 6-inch nail spacing on the interior of roof sheathing panels. In lower wind regions of the United States, a standard nail spacing applies (i.e., 6 inches on panel edges and 12 inches in the panel field); refer to Table 7.1.

EXAMPLE 7.2 Roof-to-Wall Connections

Given

- Design wind speed is 120 mph gust with an open coastal exposure
- One-story home with a hip roof (28 ft clear span trusses with 2 ft overhangs)
- Roof slope is 6:12
- Trusses are spaced at 24 in on center

Find

1. Uplift and transverse shear load at the roof-to-wall connection
2. Connection detail to resist the design loads

Solution

1. Determine the design loads on the connection (Chapter 3)

Dead load (Section 3.3)

Roof dead load = 15 psf (Table 3.2)
Dead load on wall = (15 psf)[0.5(28 ft) + 2 ft] = 240 plf

Wind load (Section 3.6)

Step 1: Basic velocity pressure = 18.8 psf (Table 3.7)
Step 2: Adjust for open exposure = 1.4(18.8 psf) = 26.3 psf
Step 3: Skip
Step 4: Roof uplift G_{cp} = -0.8
Overhang G_{cp} = +0.8
Step 5: Roof uplift pressure = -0.8(26.3 psf) = -21 psf
Overhang pressure = 0.8 (26.3 psf) = 21 psf

Determine the wind uplift load on the wall.

Design load on wall = $0.6D + W_u$ (Table 3.1)
= 0.6 (240 plf) + {(-21 psf)[0.5(28 ft) + 2 ft] – (21 psf)(2 ft)}
= - 234 plf (upward)

Design load per wall-to-truss connection = (2 ft)(-234 plf) = -468 lb (upward)

Determine the transverse shear (lateral) load on the roof-to-wall connection. The transverse load is associated with the role of the roof diaphragm in supporting and transferring lateral loads from direct wind pressure on the walls.

Design lateral load on the wall-to-truss connection
= 1/2 (wall height)(wall pressure)(truss spacing)

Adjusted velocity pressure = 26.3 psf
Wall GC_p = -1.2,+1.1*
Wind pressure = 1.1(26.3 psf) = 29 psf

*The 1.1 coefficient is used since the maximum uplift on the roof and roof overhang occurs on a windward side of the building (i.e., positive wall pressure).

= 1/2 (8 ft)(29 psf)(2 ft)
= 232 lb

Thus, roof-to-wall connection combined design loads are:

468 lb (uplift)
232 lb (lateral, perpendicular to wall)*

*The lateral load parallel to a wall is not considered to be significant in this example problem, although it may be checked to verify the transfer of lateral wind loads on the roof to shear walls; refer to Chapter 6.

2. Determine a roof-to-wall connection detail to resist the combined design load.

Generally, manufacturers publish loading data for metal connectors for multiple loading directions. The designer should verify that these values are for simultaneous multi-directional loading or make reasonable adjustments as needed. In this example problem, the NDS will be used to design a simple roof tie-down strap and slant nail connection. A tie down strap will be used to resist the uplift load and typical slant nailing will be used to resist the lateral load. The slant nailing, however, does not contribute appreciably to the uplift capacity when a strap or metal connector is used; refer to Section 7.3.6.

Uplift load resistance

Assuming an 18g (minimum 0.043 inches thick) metal strap is used, determine the number of 6d common nails required to connect the strap to the truss and to the wall top plate to resist the design uplift load.

The nail shear capacity is determined as follows:

$Z = 60$ lb (NDS Table 12.3F)
$Z' = ZC_D$ (Section 7.3.2)
$= (60 \text{ lb})(1.6)$
$= 96$ lb

The number of nails required in each end of the strap is

$(486 \text{ lb})/(96 \text{ lb/nail}) = 5$ nails

The above Z value for metal side-plates implicitly addresses failure modes that may be associated with strap/nail head tear-through. However, the width of the strap must be calculated. Assuming a minimum 33 ksi steel yield strength and a standard 0.6 safety factor, the width of the strap is determined as follows:

$0.6(33{,}000 \text{ psi})(0.043 \text{ in})(w) = 468$ lb

$w = 0.55$ in

Therefore, use a minimum 1-inch wide strap to allow for the width of nail holes and an a staggered nail pattern. Alternatively, a thinner strap may be used (i.e., 20g or 0.033 inches thick) which may create less problem with installing finishes over the connection.

Lateral load resistance

Assuming that a 16d pneumatic nail will be used (0.131 in diameter by 3.5 inches long), determine the number of slant-driven nails required to transfer the lateral load from the wall to the roof sheathing diaphragm through the roof trusses. Assume that the wall framing is Spruce-Pine-Fir ($G = 0.42$).

Z = 88 lb	(NDS Table 12.3A)*
	*A 1-1/4- inch side member thickness is used to account for the slant nail penetration through the truss at an angle.
$Z' = ZC_D$**	**The C_{tn} value of 0.83 is not used because the nail is slant driven and is not a toe-nail; refer to Section 7.3.6.

$Z' = (88 \text{ lb})(1.6) = 141 \text{ lb}$

Therefore, the number of nails required to transfer the transverse shear load is determined as follows:

(232 lb)/(141 lb/nail) = 2 nails

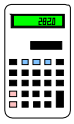
Conclusion

The beginning of the uplift load path is on the roof sheathing which is transferred to the roof framing through the sheathing nails; refer to Example 7.1. The uplift load is then passed through the roof-to-wall connections as demonstrated in this example problem. It should be noted that the load path for wind uplift cannot overlook any joint in the framing.

One common error is to attach small roof tie-straps or clips to only the top member of the wall top plate. Thus, the uplift load must be transferred between the two members of the double top plate which are usually only face nailed together for the purpose of assembly, not to transfer large uplift loads. This would not normally be a problem if the wall sheathing were attached to the top member of the double top plate, but walls are usually built to an 8 ft – 1 in height to allow for assembly of interior finishes and to result in a full 8 ft ceiling height after floor and ceiling finishes. Since sheathing is a nominal 8 ft in length, it cannot span the full wall height and may not be attached to the top member of the top plate. Also, the strap should be placed on the structural sheathing side of the wall unless framing joints within the wall (i.e., stud-to-plates) are adequately reinforced.

Longer sheathing can be special ordered and is often used to transfer uplift and shear loads across floor levels by lapping the sheathing over the floor framing to the wall below. The sheathing may also be laced at the floor band joist to transfer uplift load, but the cross grain tension of the band joist should not exceed a suitably low stress value (i.e., $1/3F_v$); refer to Chapter 5, Section 5.3.1.

EXAMPLE 7.3 Rafter-to-Ceiling Joist Connection (Heel Joint)



Given

- Rafter and ceiling joist roof construction (without intermediate rafter braces)
- Roof horizontal span is 28 ft and rafter slope is 6:12 (26 degrees)
- Roof framing is Hem-Fir (G=0.43) with a spacing of 16 inches on-center
- Roof snow load is 25 psf
- Rafter & roofing dead load is 10 psf
- Ceiling dead load is 5 psf

Find

1. The tension load on the heel joint connection
2. Nailing requirements

Solution

1. Determine the tensile load on the heel joint connection

Using basic principles of mechanics and pinned-joint analysis of the rafter and ceiling joist "truss" system, the forces on the heel joint can be determined. First, the rafter bearing reaction is determined as follows:

B = (snow + dead load)(1/2 span)(rafter spacing)
= (25 psf + 10 psf)(14 ft)(1.33 ft)
= 652 lb

Summing forces in the y-direction (vertical) for equilibrium of the heel joint connection, the compression (axial) force in the rafter is determined as follows:

$C = (652 \text{ lb})/\sin(26^o) = 1{,}487 \text{ lb}$

Now, summing the forces in the x-direction (horizontal) for equilibrium of the heel joint connection, the tension (axial) force in the ceiling joist is determined as follows:

$T = (1{,}487 \text{ lb})\cos(26^o) = 1{,}337 \text{ lb}$

2. Determine the required nailing for the connection

Try a 12d box nail. Using NDS Table 12.3A, the following Z value is obtained:

Z = 80 lb
$Z' = ZC_DC_d$ (Section 7.3.2)

C_D = 1.25* (snow load duration, Table 5.3)
*NDS uses a factor of 1.15

$C_d = p/(12D)$ (NDS•12.3.4)
p = penetration into main member = 1.5 inches
D = nail diameter = 0.128 inches
$C_d = 1.5/[12(0.128)] = 0.98$

$Z' = (80 \text{ lb})(1.25)(0.98) = 98 \text{ lb}$

In Section 5.6.1, a system factor of 1.1 for tension members and connections in trussed, light-frame roofing systems was discussed for repetitive member applications (i.e., framing spaced no greater than 24 inches on center). Therefore, the Z' value may be adjusted as follows:

$Z' = (98 \text{ lb})(1.1) = 108 \text{ lb}$

The total number of 12d box nails required is determined as follows:

$(1{,}337 \text{ lb})/(108 \text{ lb/nail}) = 12.3$

If a 16d common nail is substituted, the number of nails may be reduced to about 8. If, in addition, the species of framing lumber was changed to Southern Yellow Pine (G = 0.55), the number of nails could be reduced to 6.

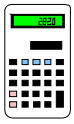
Conclusion

This example problem demonstrates the design of one of the most critical roof framing connections for site-built rafter and ceiling joist framing. In some cases, the ceiling joist or cross-tie may be located at a higher point on the rafter than the wall bearing location which will increase the load on the joint. In most designs, a simple pinned-joint analysis of the roof framing is used to determine the connection forces for various roof framing configurations.

The snow load duration factor of 1.25 was used in lieu of the 1.15 factor recommended by the NDS; refer to Table 5.3. In addition, a system factor for repetitive member, light-frame roof systems was used. The 1.1 factor is considered to be conservative which may explain the difference between the design solution in this example and the nailing required in Table 7.1 by conventional practice (i.e., four 16d common nails). If the slant nailing of the rafter to the wall top plate and wall top plate to the ceiling joist are considered in transferring the tension load, then the number of nails may be reduced relative to that calculated above. If a larger system factor than 1.1 is considered (say 1.3), then the analysis will become more closely aligned with conventional practice; refer to the roof framing system effects discussion in Section 5.6.1. It should also be remembered that the NDS safety factor on nail lateral capacity is generally in the range of 3 to 5. However, in more heavily loaded conditions (i.e., lower roof slope, higher snow load, etc.) the connection design should be expected to depart somewhat from conventional practice that is intended for “typical” conditions of use.

In any event, 12 nails per rafter-ceiling joist joint may be considered unacceptable by some builders and designers since the connection is marginally “over-crowed” with fasteners. Therefore, alternative analysis methods and fastener solutions should be considered with some regard to extensive experience in conventional practice; refer to NDS•7.1.1.4 and the discussion above.

EXAMPLE 7.4 Wall Sole Plate to Floor Connection



Given

- A 2x4 wall bottom (sole) plate of Spruce-Pine-Fir is fastened to a wood floor deck
- Floor framing lumber is Hem-Fir
- A 3/4-inch-thick wood structural panel subfloor is used
- The bottom plate is subject to the following design loads due to wind and/or earthquake lateral loads:
  250 plf shear parallel-to-grain (shear wall slip resistance)
  120 plf shear perpendicular-to-grain (transverse load on wall)
- The uplift load on the wall, if any, is assumed to be resisted by other connections (i.e., uplift straps, shear wall hold-downs, etc.)

Find A suitable nailing schedule for the wall sole plate connection using 16d pneumatic nails (0.131inch diameter by 3.5 inches long).

Solution

It is assumed that the nails will penetrate the sub-flooring and the floor framing members. It will also be conservatively assumed that the density of the sub-floor sheathing and the floor framing is the same as the wall bottom plate (lowest density of the connected materials). These assumptions allow for the use of NDS Table 12.3A. Alternatively, a more accurate nail design lateral capacity may be calculated using the yield equations of NDS•12.3.1.

Using NDS Table 12.3A, it is noted that the closest nail diameters in the table are 0.135 and 0.128 inches. Interpolating between these values, using a side member thickness of 1.5 inches, and assuming Spruce-Pine-Fir for all members, the following Z value is obtained:

$$Z = 79 + [(0.131-0.128)/(0.135-0.128)](88\text{ lb} - 79\text{ lb}) = 83\text{ lb}^*$$

$$Z' = ZC_D = 83\text{ lb}\ (1.6) = 133\text{ lb}$$

*Using the NDS general dowel equations as presented in AF&PA Technical Report 12 (AF&PA, 1999), the calculated value is identical under the same simplifying assumptions. However, a higher design value of 90 pounds may be calculated by using only the subfloor sheathing as a side member with G = 0.5. The ultimate capacity is conservatively predicted as 261 pounds.

Assuming that both of the lateral loads act simultaneously at their full design value (conservative assumption), the resultant design load is determined as follows:

$$\text{Resultant shear load} = \text{sqrt}[(250\text{plf})^2 + (120\text{ plf})^2] = 277\text{ plf}$$

Using the conservative assumptions above, the number of nails per linear foot of wall plate is determined as follows:

$$(277\text{ lb})/(133\text{ lb/nail}) = 2.1\text{ nails per foot}$$

Rounding this number, the design recommendation is 2 nails per foot or 3 nails per 16 inches of wall plate.

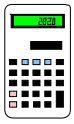
Conclusion

The number of 16d pneumatic nails (0.131 inch diameter) required is 2 nails per foot of wall bottom plate for the moderate loading condition evaluated. The number of nails may be reduced by using a larger diameter nail or by evaluating the nail lateral capacity using the yield equations of NDS•12.3.1.

As in Example 7.3, some consideration of extensive experience in conventional residential construction should also be considered in view of the conventional fastening requirements of Table 7.1 for wood sole plate to floor framing connections (i.e., one 16d nail at 16 inches on center); refer to NDS•7.1.1.4. Perhaps 2 nails per 16 inches on center is adequate for the loads assumed in this example problem. Testing has indicated that the ultimate capacity of 2 16d pneumatic nails (0.131 inch diameter) can exceed 600 lb per nail for conditions similar to those assumed in this example problem; refer to Section 7.3.6. The general dowel equations under predict the ultimate capacity by about a factor of two. Using 2 16d pneumatic nails at 16 inches on center may be expected to provide a safety factor of greater than 3 relative to the design lateral load assumed in this problem (i.e., [600 lb/nail] x [2nails/1.33 ft]/277 plf = 3.2).

As noted in Chapter 6, the ultimate capacity of base connections for shear walls should at least exceed the ultimate capacity of the shear wall for seismic design and, for wind design, the connection should at least provide a safety factor of 2 relative to the wind load. For seismic design, the safety factor for shear walls recommended in this guide is 2.5; refer to Chapter 6, Section 6.5.2.3. Therefore, the fastening schedule of 2-16d pneumatic nails at 16 inches on center is not quite adequate for seismic design loads of the magnitude assumed in this problem (i.e., the connection does not provide a safety factor of at least 2.5). The reader is referred to Chapter 3, Section 3.8.4 for additional discussion on seismic design considerations and the concept of “balanced” design.

EXAMPLE 7.5 Side-Bearing Joist Connection



Given

- A 2x10 Douglas-Fir joist is side-bearing (shear connection) on a built-up wood girder
- The design shear load on the side-bearing joint is 400 lb due to floor live and dead loads

Find

1. The number of 16d box toenails required to transfer the side-bearing (shear) load.
2. A suitable joist hanger

Solution

1. Determine the number of 16d box toenails required

$$Z' = ZC_DC_dC_{tn}$$

$Z = 103$ lb	(NDS Table 12.3A)
$C_D = 1.0$	(normal duration load)
$C_d = 1.0$	(penetration into main member > 12D)
$C_{tn} = 0.83$	(NDS•12.3.7)

$Z' = (103 \text{ lb})(0.83) = 85$ lb

The number of toenails required is determined as follows:

(400 lb)/(85 lb/nail) = 4.7 nails

Use 6 toenails with 3 on each side of the joist to allow for reasonable construction tolerance in assembling the connection in the field.

2. As an alternative, select a suitable manufactured joist hanger.

Data on metal joist hangers and various other connectors are available from a number of manufacturers of these products. The design process simply involves the selection of a properly rated connector of the appropriate size and configuration for the application. Rated capacities of specialty connectors are generally associated with a particular fastener and species of framing lumber. Adjustments may be necessary for use with various lumber species and fastener types.

Conclusion

The example problem details the design approach for two simple methods of transferring shear loads through a side-bearing connection. One approach uses a conventional practice of toe-nailing the joist to a wood girder. This approach is commonly used for short-span floor joists (i.e., tail joist to header joist connections at a floor stairwell framing). For more heavily loaded applications, a metal joist hanger is the preferred solution.

EXAMPLE 7.6 Wood Floor Ledger Connection to a Wood or Concrete Wall

Given

- A 3x8 wood ledger board (Douglas-Fir) is used to support a side-bearing floor system.
- The ledger is attached to 3x4 wall studs (Douglas-Fir) spaced at 16 inches on center in a balloon-framed portion of a home; as a second condition, the ledger is attached to a concrete wall.
- The design shear load on the ledger is 300 plf due to floor live and dead loads.

Find

1. The spacing of 5/8-inch-diameter lag screws required to fasten the ledger to the wood wall framing
2. The spacing of 5/8-inch-diameter anchor bolts required to fasten the ledger to a concrete wall

Solution

1. Determine connection requirements for use of a 5/8-inch-diameter lag screw

$Z' = ZC_DC_gC_\Delta C_d$ (Section 7.3.2)

$Z_{s\perp} = 630$ lb* (NDS Table 9.3A)
$C_D = 1.0$ (normal duration load)
$C_g = 0.98$ (2 bolts in a row) (NDS Table 7.3.6A)
$C_\Delta = 1.0$**
$C_d = p/(8D) = (3.09 \text{ in})/[8(5/8 \text{ in})] = 0.62$ (NDS•9.3.3)
p = (penetration into main member) – (tapered length of tip of lag screw)***
= 3.5 in – 13/32 in = 3.09 in

*The $Z_{s\perp}$ value is used for joints when the shear load is perpendicular to the grain of the side member (or ledger in this case).
**A C_Δ value of 1.0 is predicated on meeting the minimum edge and end distances required for lag screws and bolts; refer to NDS•8.5.3 and NDS•9.4. The required edge distance in the side member is 4D from the top of the ledger (loaded edge) and 1.5D from the bottom of the ledger (unloaded edge), where D is the diameter of the bolt or lag screw. The edge distance of 1.5D is barely met for the nominal 3-inch-wide (2.5 inch actual) stud provided the lag screws are installed along the center line of the stud.
***A 6-inch-long lag screw will extend through the side member (2.5 inches thick) and penetrate into the main member 3.5 inches. The design penetration into the main member must be reduced by the length of the tapered tip on the lag screw (see Appendix L of NDS for lag screw dimensions).

$Z' = (630 \text{ lb})(1.0)(0.98)(1.0)(0.62) = 383$ lb

The lag bolt spacing is determined as follows:

Spacing = (383 lb/lag screw)/(300 plf) = 1.3 ft

Therefore, one lag screw per stud-ledger intersection may be used (i.e., 1.33 ft spacing). The lag screws should be staggered about 2 inches from the top and bottom of the 3x8 ledger board. Since the bolts are staggered (i.e., not two bolts in a row), the value of C_g may be revised to 1.0 in the above calculations.

2. Determine connection requirements for use of a 5/8-inch-diameter anchor bolt in a concrete wall

$Z' = ZC_DC_gC_\Delta$ (Section 7.3.2)

$Z_\perp = 650$ lb* (NDS Table 8.2E)
$C_D = 1.0$ (normal duration load)
$C_g = 1.0$**
$C_\Delta = 1.0$***

* The $Z_\perp$ value is used since the ledger is loaded perpendicular to grain
**The bolts will be spaced and staggered, not placed in a row.
***Edge and end distance requirements of NDS•8.5.3 and NDS•8.5.4 will be met for full design value.

$Z' = (650 \text{ lb})(1.0)(1.0)(1.0) = 650 \text{ lb}$

The required anchor bolt spacing is determined as follows:

Spacing = (650 lb)/(300 plf) = 2.2 ft

Therefore, the anchor bolts should be spaced at about 2 ft on center and staggered from the top and bottom edge of the ledger by a distance of about 2 inches.

Note: In conditions where this connection is also required to support the wall laterally (i.e., an outward tension load due to seismic loading on a heavy concrete wall), the tension forces may dictate additional connectors to transfer the load into the floor diaphragm. In lower wind or seismic load conditions, the ledger connection to the wall and the floor sheathing connection to the ledger are usually sufficient to transfer the design tension loading, even though it may induce some cross grain tension forces in the ledger. The cross-grain tension stress may be minimized by locating every other bolt as close to the top of the ledger as practical or by using a larger plate washer on the bolts.

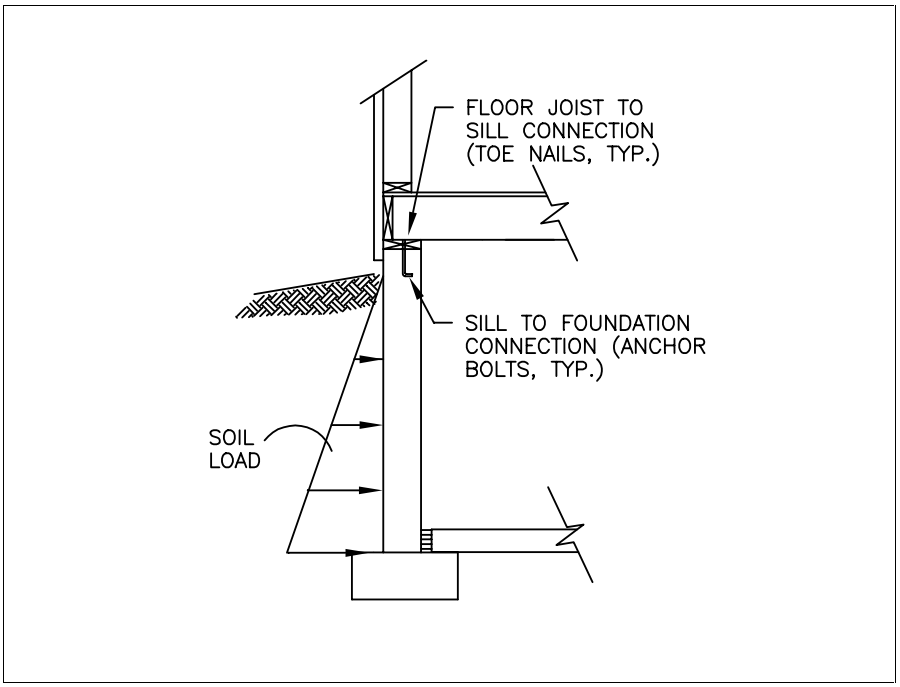
Conclusion

The design of bolted side-bearing connections was presented in this design example for two wall construction conditions. While not a common connection detail in residential framing, it is one that requires careful design consideration and installation since it must transfer the floor loads (i.e., people) through a shear connection rather than simple bearing. The example also addresses the issue of appropriate bolt location with respect to edge and end distances. Finally, the designer was alerted to special connection detailing considerations in high wind and seismic conditions.

EXAMPLE 7.7 Wood Sill to Foundation Wall

Given

- The foundation wall is connected to a wood sill plate and laterally supported as shown in the figure below.
- Assume that the soil has a 30 pcf equivalent fluid density and that the unbalanced backfill height is 7.5 ft.
- The foundation wall unsupported height (from basement slab to top of wall) is 8 ft.
- The wood sill is preservative-treated Southern Yellow Pine.



Find

1. The lateral load on the foundation wall to sill plate connection due to the backfill lateral pressure
2. The required spacing of ½-inch-diameter anchor bolts in the sill plate

Solution

1. Determine the lateral load on the sill plate connection

Using the procedure in Section 3.5 of Chapter 3 and the associated beam equations in Appendix A, the reaction at the top of the foundation wall is determined as follows:

$R_{top} = ql^3/(6L) = (30 \text{ pcf})(7.5 \text{ ft})^3/[6(8 \text{ ft})] = 264 \text{ plf}$

2. Determine the design lateral capacity of the anchor bolt and the required spacing

$Z' = ZC_DC_MC_tC_gC_\Delta$ (Section 7.3.2)

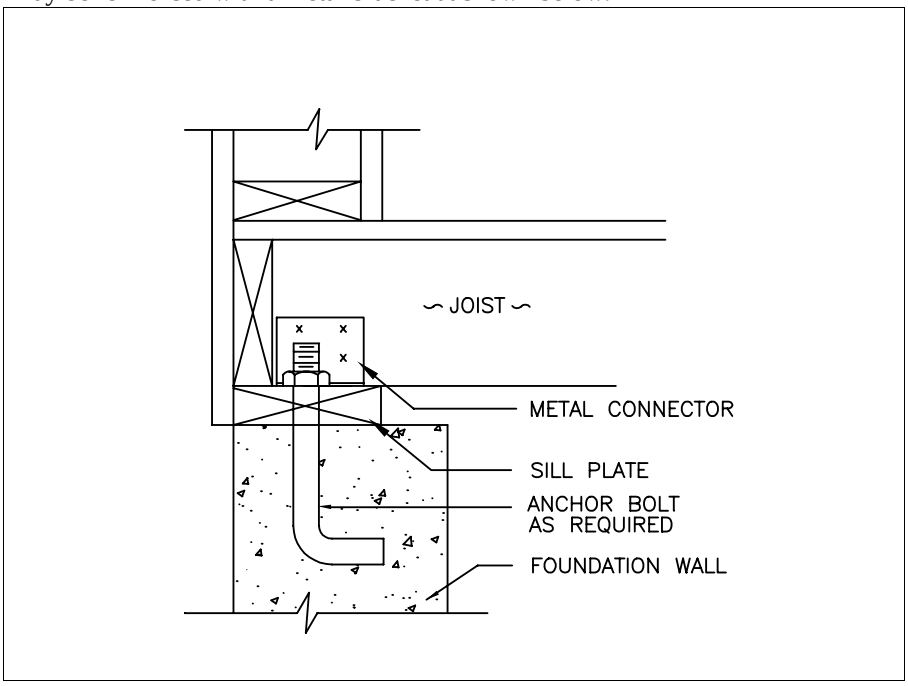
$Z_\perp = 400$ lbs* (NDS Table 8.2E)
$C_D = 0.9$ (life-time load duration, Table 5.3)
$C_M = 1.0$ (MC < 19%)
$C_t = 1.0$ (temperature < 100°F)
$C_g = 1.0$ (bolts not configured in rows)

*The value is based on a recommended 6 inch standard embedment of the anchor bolt into the concrete wall. Based on conventional construction experience, this value may also be applied to masonry foundation wall construction when bolts are properly grouted into the masonry wall (i.e., by use of a bond beam).

$Z' = (400 \text{ lb})(0.9) = 360 \text{ lb}$

Anchor bolt spacing = (360 lb)/(264 plf) = 1.4 ft

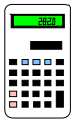
Note: According to the above calculations, an anchor bolt spacing of about 16 inches on center is required in the sill plate. However, in conventional residential construction, extensive experience has shown that a typical anchor bolt spacing of 6 ft on center is adequate for normal conditions as represented in this design example. This conflict between analysis and experience creates a dilemma for the designer that may only be reconciled by making judgmental use of the "extensive experience" clause in NDS•7.1.1.4. Perhaps a reasonable compromise would be to require the use of a 5/8-inch-diamter anchor bolt at a 4 ft on center spacing. This design may be further justified by consideration of friction in the connection (i.e., a 0.3 friction coefficient with a normal force due to dead load of the building). The large safety factor in wood connections may also be attributed to some of the discrepancy between practice or experience and analysis in accordance with the NDS. Finally, the load must be transferred into the floor framing through connection of the floor to the sill (see Table 7.1 for conventional toenail connection requirements). In applications where the loads are anticipated to be much greater (i.e., taller foundation wall with heavier soil loads), the joint may be reinforced with a metal bracket at shown below.



Conclusion

This example demonstrates an analytic method of determining foundation lateral loads and the required connections to support the top of the foundation wall through a wood sill plate and floor construction. It also demonstrates the discrepancy between calculated connection requirements and conventional construction experience that may be negotiated by permissible designer judgment and use of conventional residential construction requirements.

EXAMPLE 7.8 Deck Header to Post Connection



Given

- A 2x8 preservative-treated header is attached to each side of a deck post in a bolted, double shear connection to support load from deck joists bearing on the headers.
- The deck post is a preservative treated 4x4.
- The deck framing lumber is preservative-treated Southern Yellow Pine.
- The design double shear load on the connection is 2,560 lb (1,280 lb per header).

Find Determine if two 5/8-inch-diameter bolts are sufficient to resist the design load.

Solution

Calculate the design shear capacity of the bolted joint assuming that the bolts are located approximately 2 inches from the top and bottom edge of the 2x8 headers along the centerline of the 4x4 post.

$Z' = ZC_DC_MC_tC_gC_d$ (Section 7.3.2)

$Z_{s\perp}$ = 1,130 lb*	(NDS Table 8.3A)
C_D = 1.0**	(Normal duration of load)
C_M = 1.0	(MC < 19%)
C_t = 1.0	(Temperature < 100°F)
C_g = 0.98 (2 bolts in a row)	(NDS Table 7.3.6A)
C_Δ = 1.0 (for the bottom bolt only)***	(NDS•8.5.3)

*The $Z_{s\perp}$ value is used because the side members (2x8) are loaded perpendicular to grain and the main member (4x4) is loaded parallel to grain.

**A normal duration of load is assumed for the deck live load. However, load duration studies for deck live loads have not been conducted. Some recent research has indicated that a load duration factor of 1.25 is appropriate for floor live loads; refer to Table 5.3 of Chapter 5.

***The top bolt is placed 2 inches from the top (loaded) edge of the 2x8 header and does not meet the 4D (2.5 inch) edge distance requirement of NDS•8.5.3. However, neglecting the bolt entirely will under-estimate the capacity of the connection.

$Z' = (1{,}130 \text{ lb})(0.98) = 1{,}107 \text{ lb}$ (bottom bolt only)

If the top bolt is considered to be 80 percent effective based on its edge distance relative to the required edge distance (i.e., 2 inches / 2.5 inches = 0.8), then the design shear capacity for the two bolts in double shear may be estimated as follows:

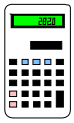
$Z' = 1{,}107 \text{ lb} + 0.8(1{,}107 \text{ lb}) = 1{,}993 \text{ lb}$ $< 2{,}560$ lb NG?

Conclusion

The calculation of the design shear capacity of a double shear bolted connection is demonstrated in this example. As shown in the calculations, the connection doesn't meet the required load in the manner analyzed. A larger bolt diameter or 3 bolts may be used to meet the required design load. However, as in previous examples, this connection is typical in residential deck construction (i.e., supporting deck spans of about 8 ft each way) and may be approved by the "extensive experience" clause of NDS•7.1.1.4. As additional rationale, the

capacity of shear connections in the NDS is related to a yield (or deformation) limit state and not capacity. On the basis of capacity, the safety margins are fairly conservative for such applications; refer to Section 7.3.1. The use of a 1.25 load duration factor for the deck live load will also increase the joint capacity to a value nearly equivalent to the design load assumed in this example.

EXAMPLE 7.9 Wood King and Jamb Stud to Floor or Foundation Connection



Given

- From Example 7.2, the net design uplift load at the roof-to-wall connection was determined to be 234 plf for a 120 mph gust, open exposure wind condition.
- Assume that the uplift loads at the top of the wall are adequately transferred through interconnection of wall framing members (i.e. top plates, sheathing, studs, headers to king and jamb studs, etc.) to the base of the upper story wall.
- The framing lumber is Hem-Fir

Find

1. The net uplift load at the base of the king and jamb studs adjacent to a 6 ft wide wall opening
2. An adequate connection detail to transfer the uplift load

Solution

1. Determine the net design uplift load at the base of the king and jamb studs supporting the 6 ft header using the ASD load combinations in Chapter 3.

Tributary load
= (1/2 header span + 1/2 stud spacing)[uplift load – 0.6(wall dead load)]
= [0.5(6 ft) + 0.5(1.33 ft)][234 plf – 0.6(64 plf)]
= 717 lb (uplift)

2. Determine the number of 8d common nails in each end of an 18g (0.043 inch minimum thickness) steel strap

$Z' = ZC_D$ (Section 7.3.2)

$Z = 82$ lb (NDS Table 12.3F)
$C_D = 1.6$ (wind load duration)

$Z' = (82 \text{ lb})(1.6) = 131$ lb

The number of nails required in each end of the strap is determined as follows:

(717 lb)/(131 lb/nail) = 6 nails

Note: As an option to the above solution, the same strap used on the layout studs may be used on the jamb and king stud connection by using multiple straps. The uplift strap on the layout studs would be required to resist 234 plf (1.33 ft) = 311 lb. Therefore, two or three of these straps could be used at wall opening location and attached to the jamb and king studs. If the single strap is used as calculated in the example problem, the jamb and king studs should be adequately interconnected (i.e., face nailed) to transfer shear load from one to the other. For example, if the header is strapped down to the top of the jamb stud and the king stud is strapped at its base, then the two members must be adequately fastened together. To some degree, the sheathing connections and other conventional connections will assist in strengthening the overall load path and their contribution should be considered or enhanced as appropriate.

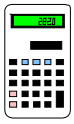
As another alternative design, the king/jamb stud uplift connection may serve a dual role as a wind uplift strap and a shear wall hold-down restraint if the wall segment adjacent to the opening is designed to be a part of the building's lateral force resisting system (i.e., shear wall segment). The method to calculate hold-down restraint forces for a shear wall is detailed in Chapter 6, Section 6.5.2.4. The uplift force due to wind would be simply added to the uplift force due to shear wall restraint to properly size a hold-down bracket or larger strap than required for wind uplift alone.

Regardless of whether or not the wall segment is intended to be a shear wall segment, the presence of wind uplift straps will provide overturning restraint to the wall such that it effectively resists shear load and creates overturning restraint forces in the uplift straps. This condition is practically unavoidable because the load paths are practically inseparable, even if the intention in the design analysis is to have separate load paths. For this reason, the opposite of the approach described in the paragraph above may be considered to be more efficient. In other words, the wind uplift strap capacity may be increased so that these multiple straps also provide multiple overturning restraints for perforated shear walls; refer to Chapter 6, Section 6.5.2.2. Thus, one type of strap or bracket can be used for the entire job to simplify construction detailing and reduce the potential for error in the field. This latter approach is applicable to seismic design (i.e., no wind uplift) and wind design conditions.

Conclusion

In this example, the transfer of wind uplift loads through wall framing adjacent to a wall opening is addressed. In addition, several alternate design approaches are noted that may optimize the design and improve construction efficiency – even in severe wind or seismic design conditions.

EXAMPLE 7.10 Concrete Wall to Footing (Shear) Connection



Given

Maximum transverse shear load on bottom of wall = 1,050 plf (due to soil)
Dead load on wall = 1,704 plf
Yield strength of reinforcement = 60,000 psi
Wall thickness = 8 inches
Assume $\mu = 0.6$ for concrete placed against hardened concrete not intentionally roughened.
$f'_c = 3{,}000$ psi

Find

- Whether a dowel or key is required to provide increased shear transfer capacity
- If a dowel or key is required, size accordingly

Solution

1. Determine factored shear load on wall due to soil load (i.e., 1.6H per Chapter 3, Table 3.1)

$V = 1{,}050$ plf
$V_u = 1.6\ (1{,}050\ \text{plf}) = 1{,}680$ plf

2. Check friction resistance between the concrete footing and wall

$V_{friction} = \mu N = \mu(\text{dead load per foot of wall})$

$= (0.6)(1{,}704\ \text{plf}) = 1{,}022\ \text{plf} < V_u = 1{,}680\ \text{plf}$

Therefore, a dowel or key is needed to secure the foundation wall to the footing.

3. Determine a required dowel size and spacing (Section 7..2 and ACI-318•5.14)

$A_{vf} = V_u / (\phi f_y \mu)$

$= (1{,}680\ \text{plf})/[(0.85)(60{,}000)(0.6)] = 0.05\ \text{in}^2$ per foot of wall

Try a No. 4 bar ($A_v = 0.20\ \text{in}^2$) and determine the required dowel spacing as follows:

$A_{vf} = A_v/S$
$0.05\ \text{in}^2/\text{lf} = (0.2\ \text{in}^2)/S$
$S = 48$ inches

Conclusion

This example problem demonstrates that for the given conditions a minimum of one No. 4 rebar at 48 inches on center is required to adequately restrict the wall from slipping. Alternatively, a key may be used or the base of the foundation wall may be laterally supported by the basement slab.

It should be noted that the factored shear load due to the soil lateral pressure is compared to the estimated friction resistance in Step 1 without factoring the friction resistance. There is no clear guideline in this matter of designer judgment.

EXAMPLE 7.11 Concrete Anchor

Given

- 1/2-inch diameter anchor bolt at 4 feet on center with a 6 inch embedment depth in an 8-inch thick concrete wall
- The bolt is an ASTM A36 bolt with f_y = 36 ksi and the following design properties for ASD; refer to AISC Manual of Steel Construction (AISC,1989):
  - F_t = 19,100 psi (allowable tensile stress)
  - F_u = 58,000 psi (ultimate tensile stress)
  - F_v = 10,000 psi (allowable shear stress)
- The specified concrete has f'_c = 3,000 psi
- The nominal design (unfactored) loading conditions are as follows:
  - Shear load = 116 plf
  - Uplift load = 285 plf
  - Dead load = 180 plf

Find Determine if the bolt and concrete are adequate for the given conditions.

Solution

1. Check shear in bolt using appropriate ASD steel design specifications (AISC, 1989) and the ASD load combinations in Chapter 3.

$$f_v = \frac{\text{shear load}}{\text{bolt area}} = \frac{116 \text{ plf } (4 \text{ ft})}{(0.196 \text{ in}^2)} = 2{,}367 \text{ psi}$$

F_v = 10,000 psi

$f_v \le F_v$ OK

2. Check tension in bolt due to uplift using appropriate ASD steel design specifications (AISC, 1989) and the appropriate ASD load combination in Chapter 3.

$$T = [(285 \text{ plf}) - 0.6 (180 \text{ plf})] (4 \text{ ft}) = 708 \text{ lb}$$

$$f_t = T/A_{bolt} = \frac{708 \text{ lb}}{0.196 \text{ in}^2} = 3{,}612 \text{ psi}$$

$f_t \le F_t$

3,612 psi < 19,100 psf OK

3. Check tension in concrete (anchorage capacity of concrete) using ACI-318•11.3 and the appropriate LRFD load combination in Chapter 3. Note that the assumed cone shear failure surface area, A_v, is approximated as the minimum of π (bolt embedment length)^2 or π (wall thickness)^2.

$$V_u = T = [1.5 (285 \text{ plf}) - 0.9 (180 \text{ plf})] (4 \text{ ft}) = 1{,}062 \text{ lb}$$

$$A_v = \text{minimum of } \begin{cases} \pi (6 \text{ in})^2 = 113 \text{ in}^2 \\ \pi (8 \text{ in})^2 = 201 \text{ in}^2 \end{cases}$$

$$\phi V_c = \phi 4 A_v \sqrt{f'_c} = (0.85)(4)(113 \text{ in}^2)\sqrt{3{,}000 \text{ psi}} = 21{,}044 \text{ lb}$$

$V_u \le \phi V_c$

1,062 lb ≤ 21,044 lb OK

Conclusion

A 1/2-inch diameter anchor bolt with a 6 inch concrete embedment and spaced 4 feet on center is adequate for the given loading conditions. In lieu of using an anchor bolt, there are many strap anchors that are also available. The strap anchor manufacturer typically lists the embedment length and concrete compressive strength required corresponding to strap gauge and shear and tension ratings. In this instance, a design is not typically required–the designer simply ensures that the design loads do not exceed the strap anchor's rated capacity.

7.6 References

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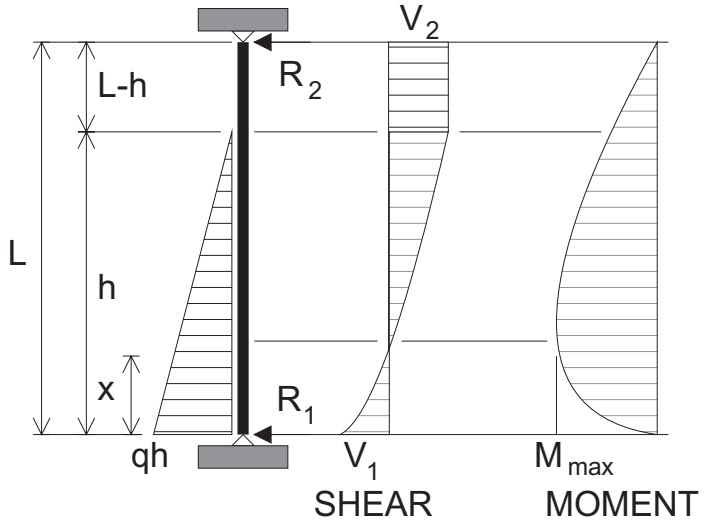
Appendix A
Shear and Moment Diagrams and Beam Equations



q = equivalent fluid density of soil (pcf)

qh = soil presure (psf) at x = 0

$$V_2 = -R_2 = \frac{-qh^3}{6L}$$

$$V_1 = R_1 = \frac{1}{2}qh^2\ (1-\frac{h}{3L})$$

$$V_x = V_1 - \frac{1}{2}xq\,(2h-x) \text{ (where } x < h)$$

$$V_x = V_2 \text{ (where } x \geq h)$$

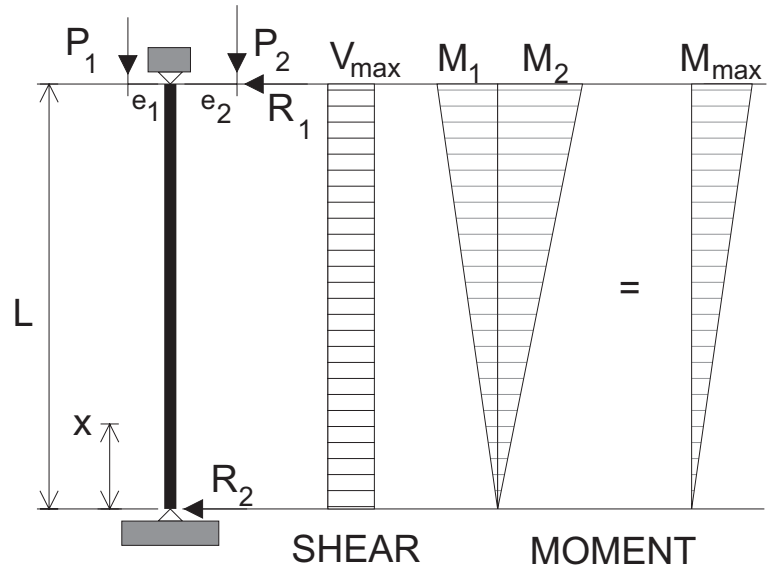
$$M_x = V_1x - \frac{1}{2}qhx^2 + \frac{1}{6}qx^3 \text{ (where } x < h)$$

$$M_x = -V_2\,(L - x) \text{ (where } x \geq h)$$

$$x_{@M_{max}} = h - \sqrt{h^2 - \frac{2V_1}{q}}$$

$$\Delta_{max}\ (\text{at } x \cong L/2) \cong \frac{qL^3}{EI}\left[\frac{hL}{128} - \frac{L^2}{960} - \frac{h^2}{48} + \frac{h^3}{144L}\right]$$

Figure A.1 - Simple Beam (Foundation Wall) - Partial Triangular Load



$$V_{max} = R_2 = \frac{M_{max}}{L}$$

$$M_1 = P_1 e_1$$

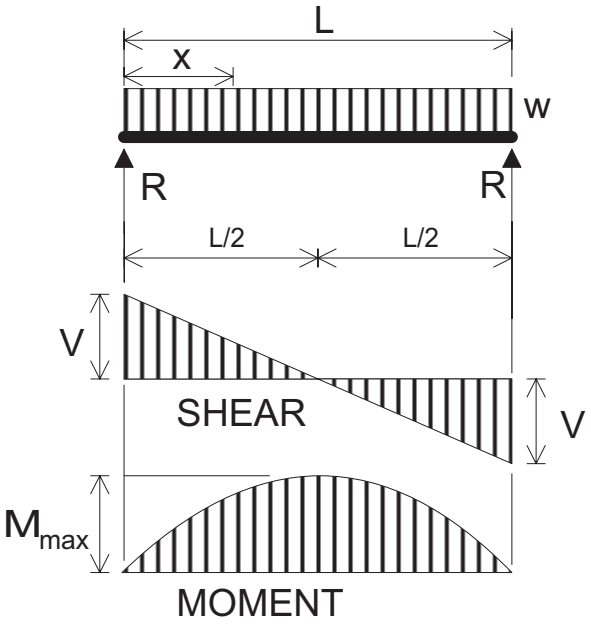
$$M_2 = P_2 e_2$$

$$M_{max} = |M_2| - |M_1| \text{ where } |M_2| > |M_1|$$

$$M_{max} = |M_1| - |M_2| \text{ where } |M_1| > |M_2|$$

$$M_x = M_{max}\left(\frac{x}{L}\right)$$

Figure A.2 - Simple Beam (Wall or Column) - Eccentric Point Loads



$$R = V_{max} = \frac{wL}{2}$$

$$V_x = w\left(\frac{L}{2} - x\right)$$

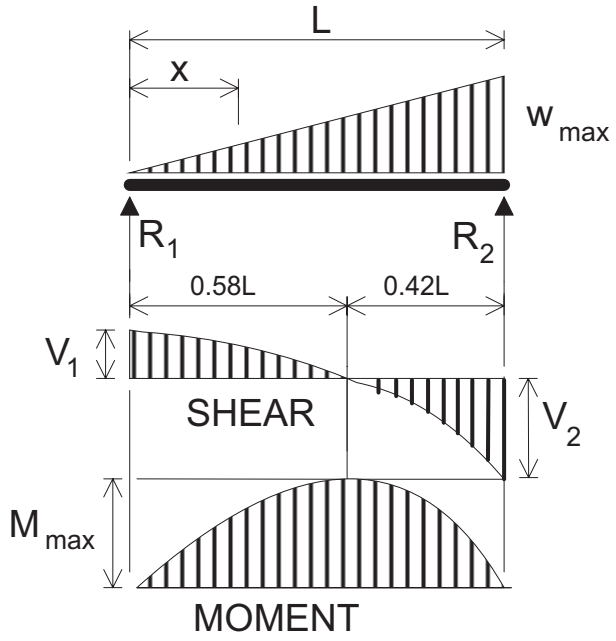
$$M_{max} = \frac{wL^2}{8} \left(\text{at } x = \frac{L}{2}\right)$$

$$M_x = \frac{wx}{2}(L - x)$$

$$\Delta_{max} = \frac{5wL^4}{384EI} \left(\text{at } x = \frac{L}{2}\right)$$

$$\Delta_x = \frac{wx}{24EI}\left(L^3 - 2Lx^2 + x^3\right)$$

Figure A.3 - Simple Beam - Uniformly Distributed Load



$$R_1 = V_1 = \frac{w_{max}}{3}$$

$$R_2 = V_2 = \frac{2w_{max}}{3}$$

$$V_x = \frac{w_{max}}{3} - \frac{w_{max}^{\ 2}}{L^2}$$

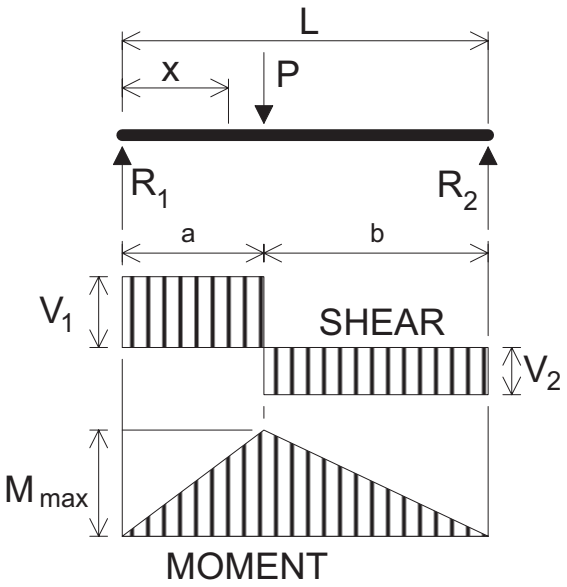
$$M_{max}\ (\text{at } x = \frac{L}{\sqrt{3}}) = \frac{2w_{max}L}{9\sqrt{3}}$$

$$M_x = \frac{w_{max}x}{3L^2}(L^2 - x^2)$$

$$\Delta_{max}\ (\text{at } x = L\sqrt{1-\sqrt{\frac{8}{15}}}) = \frac{w_{max}L^3}{77EI}$$

$$\Delta_x = \frac{w_{max}x}{180EIL^2}(3x^4 - 10L^2x^2 + 7L^4)$$

Figure A.4 - Simple Beam - Load Increasing Uniformly to One End



$R_1 = V_1$ (max when a<b) = $\frac{Pb}{L}$

$R_2 = V_2$ (max when a>b) = $\frac{Pa}{L}$

M_{max} (at point of load) = $\frac{Pab}{L}$

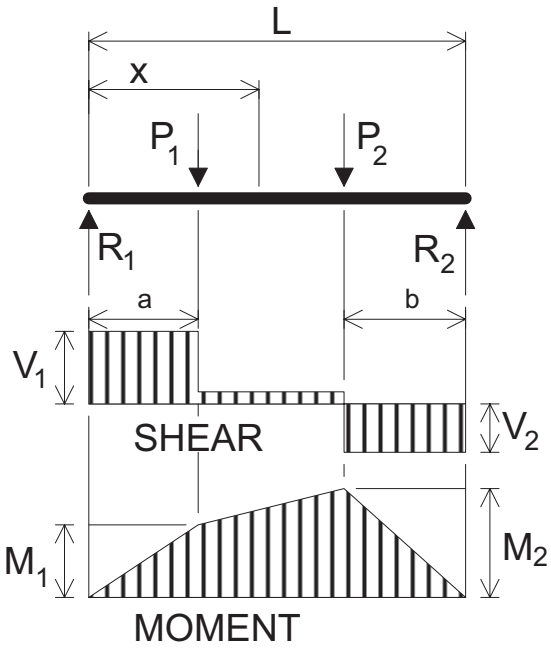
M_x (when x<a) = $\frac{Pbx}{L}$

Δ_{max} [at x = $\sqrt{\frac{a(a+2b)}{3}}$ when a<b] = $\frac{Fab(a+2b)\sqrt{3a(a+2b)}}{27EIL}$

Δ_a (at point of load) = $\frac{Pa^2b^2}{3EIL}$

Δ_x (when x<a) = $\frac{Pbx}{6EIL}(L^2 - b^2 - x^2)$

Figure A.5 - Simple Beam - Concentrated Load at Any Point



$$R_1 = V_1 = \frac{P_1 (L - a) + P_2 b}{L}$$

$$R_2 = V_2 = \frac{P_1 a + P_2 (L - b)}{L}$$

V_x [when $a<x<(L-b)$] $= R_1 - P_1$

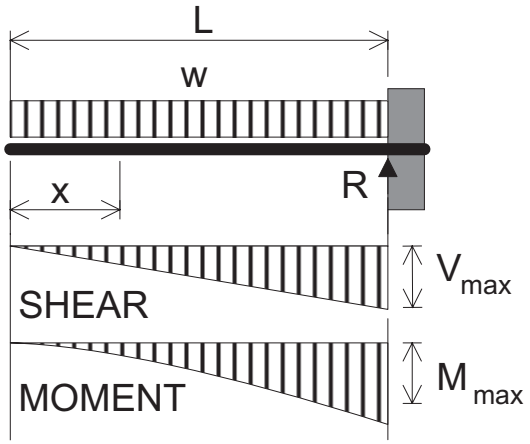
M_1 (max when $R_1<P_1$) $= R_1 a$

M_2 (max when $R_2<P_2$) $= R_2 b$

M_x (when $x<a$) $= R_1 x$

M_x [when $a<x<(L-b)$] $= R_1 x - P_1 (x-a)$

Figure A.6 - Simple Beam - Two Unequal Concentrated Loads Unsymmetrically Placed



$R = V_{max} = wL$

$V_x = wx$

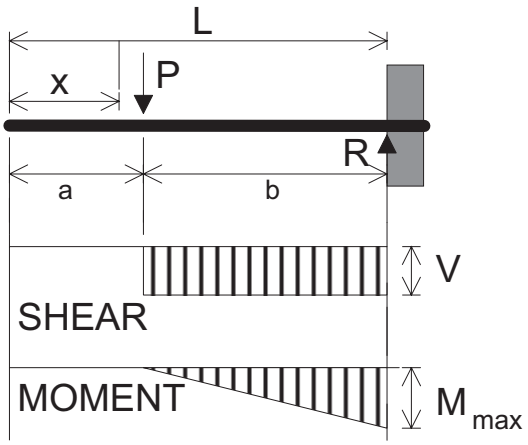
M_{max} (at fixed end) $= \frac{wL^2}{2}$

$M_x = \frac{wx^2}{2}$

Δ_{max} (at free end) $= \frac{wL^4}{8EI}$

$\Delta_x = \frac{w}{24EI} (x^4 - 4L^3 x + 3L^4)$

Figure A.7 - Cantilever Beam - Uniformly Distributed Load



$R = V = P$

M_{max} (at fixed end) = Pb

M_x (when $x>a$) = $P(x-a)$

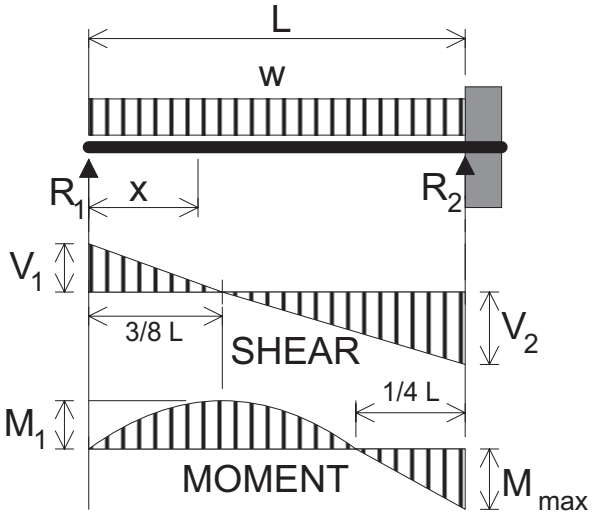
Δ_{max} (at free end) = $\frac{Pb^2}{6EI}(3L-b)$

Δ_a (at point of load) = $\frac{Pb^3}{3EI}$

Δ_x (when $x<a$) = $\frac{Pb^2}{6EI}(3L-3x-b)$

Δ_x (when $x>a$) = $\frac{P(L-x)^2}{6EI}(3b-L+x)$

Figure A.8 - Cantilever Beam - Concentrated Load at Any Point



$R_1 = V_1 = \frac{3wL}{8}$

$R_2 = V_2 = V_{max} = \frac{5wL}{8}$

$V_x = R_1 - wx$

$M_{max} = \frac{wL^2}{8}$

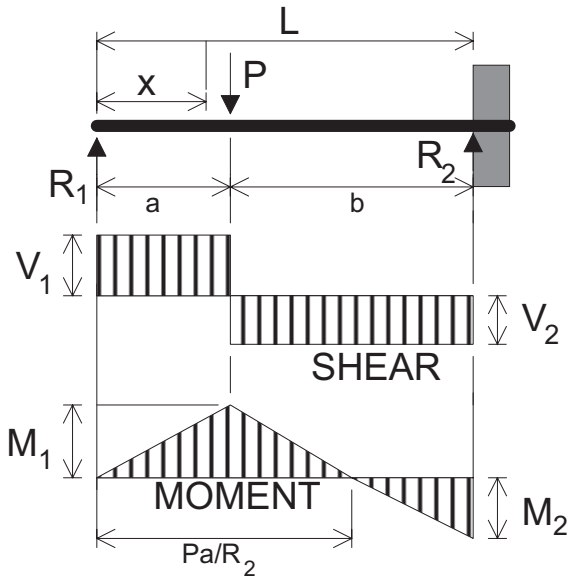
M_1 (at x =) = $\frac{3}{8}L = \frac{9}{128}wL^2$

$M_x = R_1x - \frac{wx^2}{2}$

Δ_{max} (at x = $\frac{L}{16}(1+\sqrt{33}) = 0.42L$) = $\frac{wL^4}{185EI}$

$\Delta_x = \frac{wx}{48EI}(L^3 - 3Lx^2 + 2x^3)$

Figure A.9 - Beam Fixed at One End, Supported at Other - Uniformly Distributed Load



$R_1 = V_1 = \frac{Pb^2}{2L^3}(a + 2L)$

$R_2 = V_2 = \frac{Pa}{2L^3}(3L^2 - a^2)$

M_1 (at point of load) $= R_1 a$

M_2 (at fixed end) $= \frac{Pab}{2L^2}(a + L)$

M_x (when x<a) $= R_1 x$

M_x (when x>a) $= R_1 x - P(x-a)$

Δ_{max} (when a<0.4L at x $= L\frac{L^2 + a^2}{3L^2 - a^2}$) $= \frac{Pa}{3EI}\frac{(L^2 - a^2)^3}{(3L^2 - a^2)^2}$

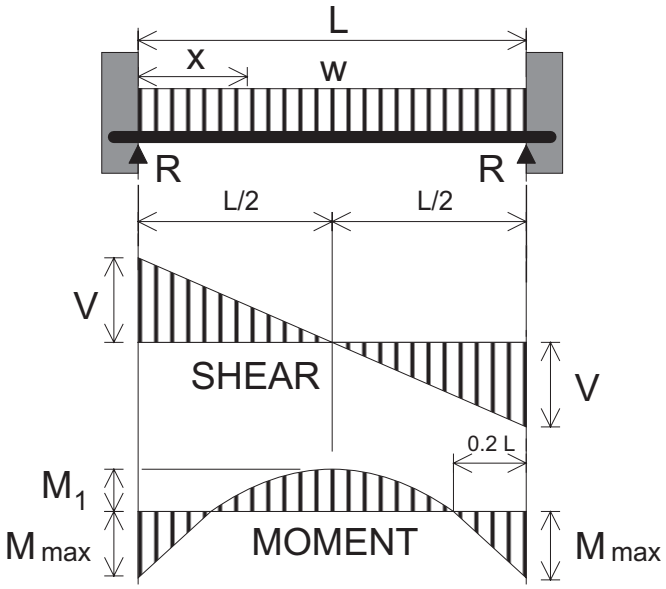
Δ_{max} (when a>0.4L at x $= L\sqrt{\frac{a}{2L + a}}$) $= \frac{Pab^2}{6EI}\sqrt{\frac{a}{2L + a}}$

Δ_a (at point of load) $= \frac{Pa^2 b^3}{12EIL^3}(3L + a)$

Δ_x (when x<a) $= \frac{Pa^2 x}{12EIL^3}(3aL^2 - 2Lx^2 - ax^2)$

Δ_x (when x>a) $= \frac{Pa}{12EIL^3}(L-x)^2(3L^2x - a^2x - 2a^2L)$

Figure A.10 - Beam Fixed at One End, Supported at Other - Concentrated Load at Any Point



$R = V = \frac{wL}{2}$

$V_x = w\left(\frac{L}{2} - x\right)$

M_{max} (at ends) $= \frac{wL^2}{12}$

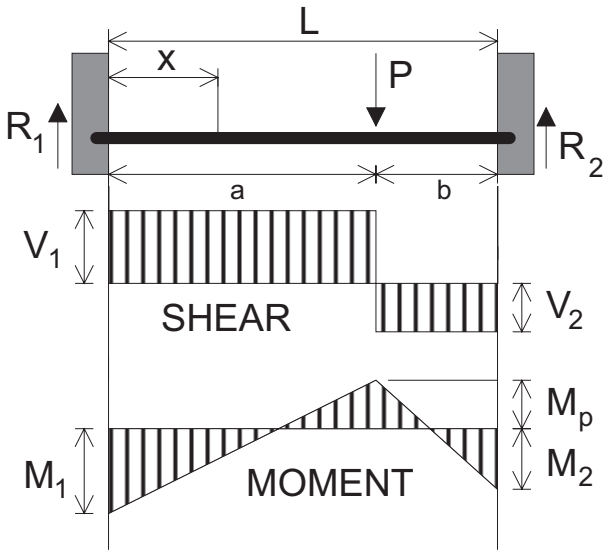
M_1 (at center) $= \frac{wL^2}{24}$

$M_x = \frac{w}{12}(6Lx - L^2 - 6x^2)$

Δ_{max} (at center) $= \frac{wL^4}{384EI}$

$\Delta_x = \frac{wx^2}{24EI}(L - x)^2$

Figure A.11 - Beam Fixed at Both Ends - Uniformly Distributed Loads



$R_1 = V_1$ (max. when a<b) = $\frac{Pb^2}{L^3}(3a + b)$

$R_2 = V_2$ (max. when a>b) = $\frac{Pa^2}{L^3}(a + 3b)$

M_1 (max. when a<b) = $\frac{Pab^2}{L^2}$

M_2 (max. when a>b) = $\frac{Pa^2b}{L^2}$

M_a (at point of load) = $\frac{2Pa^2b^2}{L^3}$

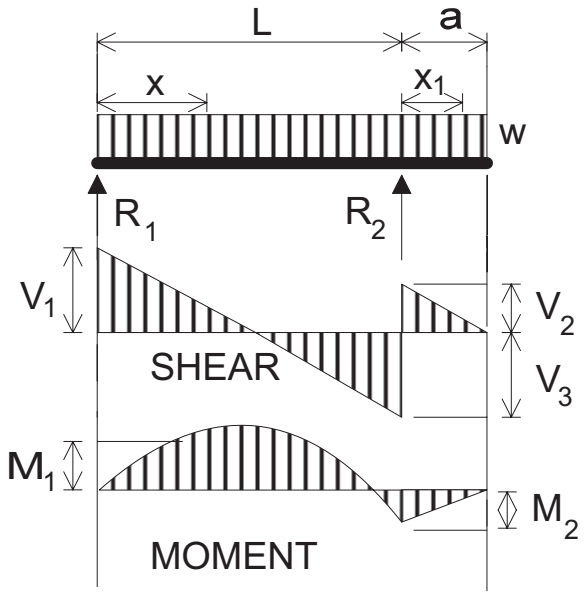
M_x (when x<a) = $R_1x - \frac{Pab^2}{L^2}$

Δ_{max} (when a>b at x = $\frac{2aL}{3a+b}$) = $\frac{2Pa^3b^2}{3EI(3a+b)^2}$

Δ_a (at point of load) = $\frac{Pa^3b^3}{3EIL^3}$

Δ_x (when x<a) = $\frac{Pb^2x^2}{6EIL^3}(3aL - 3ax - bx)$

Figure A.12 - Beam Fixed at Both Ends - Concentrated Load at Any Point



$R_1 = V_1 = \frac{w}{2L}(L^2 - a^2)$

$R_2 = V_2 + V_3 = \frac{w}{2L}(L + a)^2$

$V_2 = wa$

$V_3 = \frac{w}{2L}(L^2 + a^2)$

V_x (between supports) = $R_1 - wx$

V_{x1} (for overhang) = $w(a - x_1)$

M_1 (at x = $\frac{L}{2}\left[1 - \frac{a^2}{L^2}\right]$) = $\frac{w}{8L^2}(L+a)^2(L-a)^2$

M_2 (at R_2) = $\frac{wa^2}{2}$

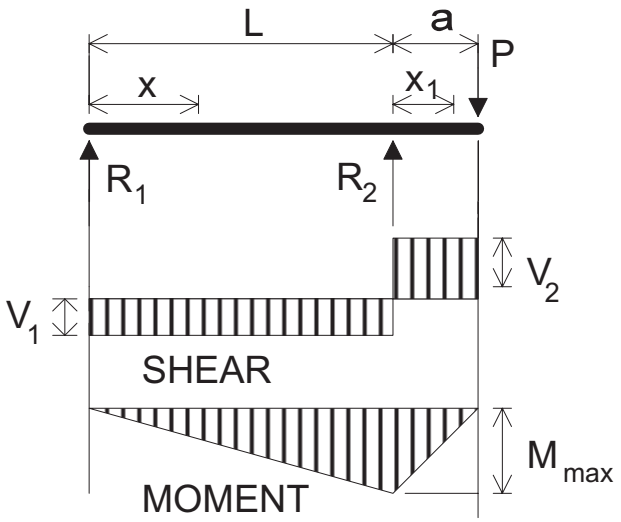
M_x (between supports) = $\frac{wx}{2L}(L^2 - a^2 - xL)$

M_{x1} (for overhang) = $\frac{w}{2}(a - x_1)^2$

Δ_x (between supports) = $\frac{24EIL}{wx}(L^4 - 2L^2x^2 + Lx^3 - 2a^2L^2 + 2a^2x^2)$

Δ_{x1} (for overhang) = $\frac{wx_1}{24EI}(4a^2L - L^3 + 6a^2x_1 - 4ax_1^2 + x_1^3)$

Figure A.13 - Beam Overhanging One Support - Uniformly Distributed Load



$R_1 = V_1 = \frac{Pa}{L}$

$R_2 = V_1 + V_2 = \frac{P}{L}(L + a)$

$V_2 = P$

M_{max} (at R_2) = Pa

M_x (between supports) = $\frac{Pax}{L}$

M_{x1} (for overhang) = $P(a - x_1)$

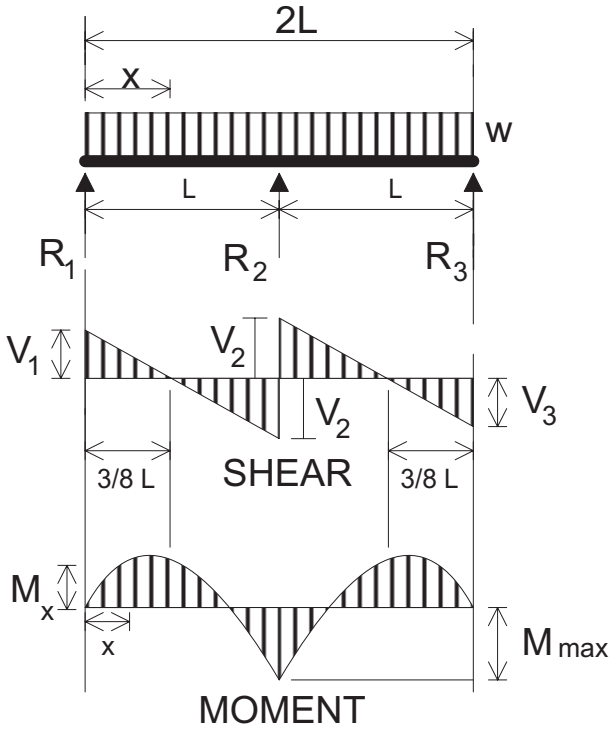
Δ_{max} (between supports at x = $\frac{L}{\sqrt{3}}$) = $\frac{PaL^2}{9\sqrt{3}EI}$

Δ_{max} (for overhang at $x_1 = a$) = $\frac{Pa^2}{3EI}(L + a)$

Δ_x (between supports) = $\frac{Pax}{6EIL}(L^2 - x^3)$

Δ_x (for overhang) = $\frac{Px_1}{6EI}(2aL + 3ax_1 - x_1^2)$

Figure A.14 - Beam Overhanging One Support - Concentrated Load at End of Overhang



$R_1 = V_1 = R_3 = V_3 = \frac{3wL}{8}$

$R_2 = \frac{10wL}{8}$

$V_2 = Vm = \frac{5wL}{8}$

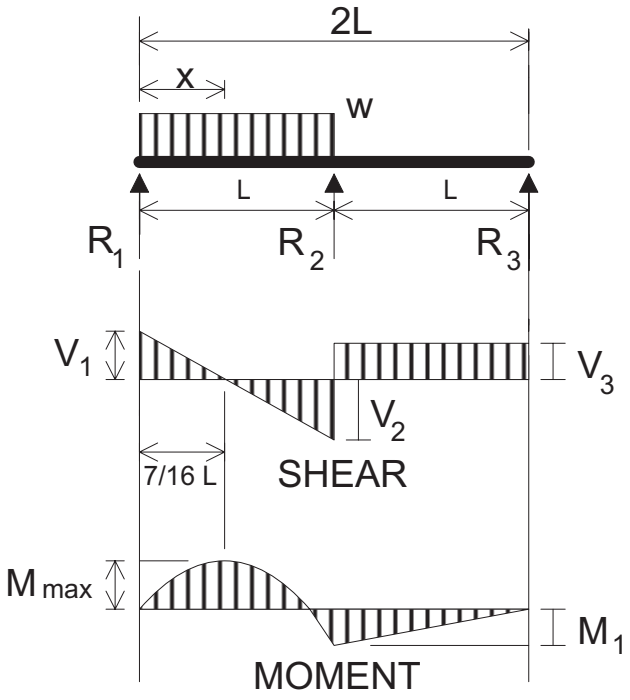
$M_{max} = -\frac{wL^2}{8}$

M_1 [at x = $\frac{3L}{8}$] = $\frac{9wL^2}{128}$

M_x [at x < L] = $\frac{3wLx}{8} - \frac{wx^2}{2}$

Δ_{max} [at x ≅ 0.46L] = $\frac{wL^4}{185EI}$

Figure A.15 - Continuous Beam - Two Equal Spans and Uniformly Distributed Load



$R_1 = V_1 = \frac{7}{16} wL$

$R_2 = V_2 + V_3 = \frac{5}{8} wL$

$R_3 = V_3 = -\frac{1}{16} wL$

$V_2 = \frac{9}{16} wL$

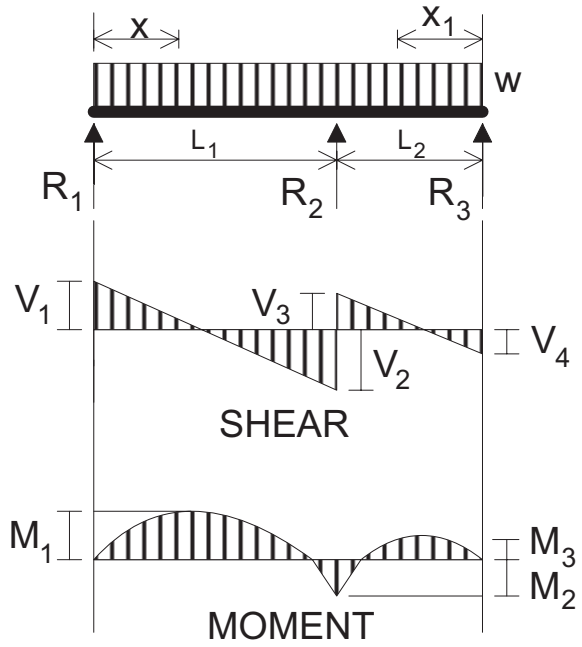
M_{max} [at $x = \frac{7}{16} L$] $= \frac{49}{512} wL^2$

M_1 [at R_2] $= -\frac{1}{16} wL^2$

M_x [at $x < L$] $= \frac{wx}{16} (7L - 8x)$

Δ_{max} [at $x \cong 0.47L$] $= \frac{wL^4}{109EI}$

Figure A.16 - Continuous Beam - Two Equal Spans with Uniform Load on One Span



$R_1 = V_1 = \frac{M_1}{L_1} + \frac{wL_1}{2}$

$R_2 = wL_1 + wL_2 - R_1 - R_3$

$R_3 = V_4 = \frac{M_1}{L_1} + \frac{wL_2}{2}$

$V_2 = wL_1 - R_1$

$V_3 = wL_2 - R_3$

M_1 [at $x < L_1$, max. at $x = \frac{R_1}{w}$] $= R_1x = \frac{wx^2}{2}$

$M_2 = -\frac{wL_2^3 + wL_1^3}{8(L_1 + L_2)}$

M_3 [at $x_1 < L_2$, max. at $x_1 = \frac{R_3}{w}$] $= R_3x_1 - \frac{wx_1^2}{2}$

Figure A.17 - Continuous Beam - Two Unequal Spans and Uniformly Distributed Load

Appendix B

Unit Conversions

The following list provides the conversion relationship between U.S. customary units and the International System (SI) units. A complete guide to the SI system and its use can be found in ASTM E 380, Metric Practice.

To convert from	to	multiply by
Length		
inch (in.)	meter(μ)	25,400
inch (in.)	centimeter	2.54
inch (in.)	meter(m)	0.0254
foot (ft)	meter(m)	0.3048
yard (yd)	meter(m)	0.9144
mile (mi)	kilometer(km)	1.6
Area		
square foot (sq ft)	square meter(sq m)	0.09290304
square inch (sq in)	square centimeter(sq cm)	6.452
square inch (sq in.)	square meter(sq m)	0.00064516
square yard (sq yd)	square meter(sq m)	0.8391274
square mile (sq mi)	square kilometer(sq km)	2.6
Volume		
cubic inch (cu in.)	cubic centimeter(cu cm)	16.387064
cubic inch (cu in.)	cubic meter(cu m)	0.00001639
cubic foot (cu ft)	cubic meter(cu m)	0.02831685
cubic yard (cu yd)	cubic meter(cu m)	0.7645549
gallon (gal) Can. liquid	liter	4.546
gallon (gal) Can. liquid	cubic meter(cu m)	0.004546
gallon (gal) U.S. liquid*	liter	3.7854118
gallon (gal) U.S. liquid	cubic meter(cu m)	0.00378541
fluid ounce (fl oz)	milliliters(ml)	29.57353
fluid ounce (fl oz)	cubic meter(cu m)	0.00002957
Force		
kip (1000 lb)	kilogram (kg)	453.6
kip (1000 lb)	Newton (N)	4,448.222
pound (lb)	kilogram (kg)	0.4535924
pound (lb)	Newton (N)	4.448222
Stress or pressure		
kip/sq inch (ksi)	megapascal (Mpa)	6.894757
kip/sq inch (ksi)	kilogram/square centimeter (kg/sq cm)	70.31

To convert from	to	multiply by
pound/sq inch (psi)	kilogram/square centimeter (kg/sq cm)	0.07031
pound/sq inch (psi)	pascal (Pa) *	6,894.757
pound/sq inch (psi)	megapascal (Mpa)	0.00689476
pound/sq foot (psf)	kilogram/square meter (kg/sq m)	4.8824
pound/sq foot (psf)	pascal (Pa)	47.88
Mass (weight)		
pound (lb) avoirdupois	kilogram (kg)	0.4535924
ton, 2000 lb	kilogram (kg)	907.1848
grain	kilogram (kg)	0.0000648
Mass (weight) per length)		
kip per linear foot (klf)	kilogram per meter (kg/m)	0.001488
pound per linear foot (plf)	kilogram per meter (kg/m)	1.488
Moment		
1 foot-pound (ft-lb)	Newton-meter (N-m)	1.356
Mass per volume (density)		
pound per cubic foot (pcf)	kilogram per cubic meter (kg/cu m)	16.01846
pound per cubic yard (lb/cu yd)	kilogram per cubic meter (kg/cu m)	0.5933
Velocity		
mile per hour (mph)	kilometer per hour (km/hr)	1.60934
mile per hour (mph)	kilometer per second (km/sec)	0.44704
Temperature		
degree Fahrenheit (°F)	degree Celsius (°C)	$t_C = (t_F-32)/1.8$
degree Fahrenheit (°F)	degree Kelvin (°K)	$t_K= (t_F+ 459.7)/1.8$
degree Kelvin (°F)	degree Celsius (°C)	$t_C = (t_K -32)/1.8$

*One U.S. gallon equals 0.8327 Canadian gallon
**A pascal equals 1000 Newton per square meter.

The prefixes and symbols below are commonly used to form names and symbols of the decimal multiples and submultiples of the SI units.

Multiplication Factor	Prefix	Symbol
$1,000,000,000 = 10^9$	giga	G
$1,000,000 = 10^6$	mega	M
$1,000 = 10^3$	kilo	k
$0.01 = 10^{-2}$	centi	c
$0.001 = 10^{-3}$	milli	m
$0.000001 = 10^{-6}$	micro	μ
$0.000000001 = 10^{-9}$	nano	n

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