Study Guide for Residential Structural Design for Home Inspectors Course

This study guide can help you:

- take notes;
- read and study offline;
- organize information; and
- prepare for assignments and assessments.

As a member of InterNACHI, you may check your education folder, transcript, and course completions by logging into your Members-Only Account at www.nachi.org/account.

To purchase textbooks (printed and electronic), visit InterNACHI's ecommerce partner Inspector Outlet at www.inspectoroutlet.com.

Copyright © 2007-2015 International Association of Certified Home Inspectors, Inc.
Student Identification & Verification

Student Verification

By enrolling in this course, the student hereby attests that they are the person completing all coursework. They understand that having another person complete the coursework for them is fraudulent and will result in being denied course completion and corresponding credit hours.

The course provider reserves the right to make contact as necessary to verify the integrity of any information submitted or communicated by the student. The student agrees not to duplicate or distribute any part of this copyrighted work or provide other parties with the answers or copies of the assessments that are part of this course. If plagiarism or copyright infringement is proven, the student will be notified of such and barred from the course and/or have their credit hours and/or certification revoked.

Communication on the message board or forum shall be of the person completing all coursework.
Introduction

The increasing complexity of homes, the use of innovative materials and technologies, and the increased population in high-hazard areas 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 home inspectors in the uniqueness of housing as a structural design problem.

This course is an 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 course may be viewed as a “living document” subject to further improvement as the art and science of housing design evolves. The desired effect is to continue to improve the value of residential housing in terms of economy and structural performance.

This course is a unique and comprehensive tool for professional home inspectors and design professionals, particularly structural engineers, seeking to provide value-added services to the producers and consumers of residential housing. As such, the course 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.

Given that most homes in the United States are built with wood structural materials, the course 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 course are based on the best information available 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 course. Therefore, a competent professional should first compare and understand any differences between the content of this course and local building code requirements. Any actual use of this information by a competent professional may require appropriate substantiation as an "alternative method of analysis." The course 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 course to steer a professional 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 course 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 information will foster a better understanding among engineers, architects, building code officials, home builders, and home inspector by clarifying the perception of homes as structural systems. As such, the course should help home inspector perform their services more effectively and assist in integrating their skills with others who contribute to the production of safe and affordable homes in North America.

**Structural Design Basics**

**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 1A through 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 framing 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 1A. Post-and-Beam Construction (Historical)

FIGURE 1B. Balloon-Frame Construction (Historical)
Conventional or prescriptive construction practices are based as much on experience as on technical analysis and theory. When incorporated into a building code, prescriptive (sometimes called "cookbook") 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.

While dimensional lumber has remained the predominant material used in 20th-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 an 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.

![Image of engineered floor joist system](image-url)
In addition, it is important to recognize that while these 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.

**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. This type of housing is constructed in essentially the same manner as site-built housing, except that the houses are plant-built in finished modules (typically two or more) and shipped to the job site 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 the 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.

**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:

**Masonry**

National Concrete Masonry Association [www.ncma.org](http://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 (OSB), glue-laminated lumber, and parallel strand lumber. 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 most new homes. The increased use of engineered wood products is the result of many years of research and product development and, more importantly, 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 job site. 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 non-standardized 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 new housing starts. The benefits of cold-formed steel include cost, durability, light weight, and
strength. Figure 1.4 illustrates the use of cold-formed steel framing in a home.

**FIGURE 1.4 House Construction Using Cold-Formed Steel Framing**

![Image from LTH Steel Structures](Image from LTH Steel Structures)

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.

**FIGURE 1.5 Insulating Concrete Forms**

Concrete masonry construction, illustrated in Figure 1.6, is essentially unchanged in basic construction methods; 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 practices. Nonetheless, at the national level, above-grade masonry wall construction represents less than 10 percent of annual housing starts.

**FIGURE 1.6 House Construction Using Concrete Masonry**
Most homes in the United States are built with ______ structural materials.

- wood
- metal
- stone
- brick

Concrete or masonry are generally assumed to be used for the ______-grade portion of the structure.

- below
- above
- mid
- non

Traditional ______ framing consists of closely spaced, light vertical structural members that extend from the foundation sill to the roof plates.

- balloon
- light
- core
- spill-form

What is the modern adaptation of balloon framing?

- platform
- sill
- post
- light
What type of roof framing is now used in most homes because it is generally more efficient than older stick-framing methods?

- Wood truss
- Metal truss
- Steel beam
- Connected truss

Most homes in the United States are built using what type of framing approach?

- Stick-framing
- Metal-framing
- Market-framing
- Joist-framing

What is the primary advantage of on-site building?

- It is flexible in meeting variations
- There are no travel costs
- It uses the least expensive materials
- It is most likely to meet code requirements

An increasing number of today’s site-built homes use components that are fabricated where?

- In off-site plants
- On site
- In site-adjacent plants

What type of housing is constructed off site in finished pieces and shipped to the job site?

- Modular housing
- Conventional housing
- Modern housing
- Manufactured housing

Modular housing is built to comply with the same building codes that govern __________ housing.

- site-built
- off site-built
- modern
- manufactured

Mobile homes can also be called:
Manufactured housing is constructed with methods that comply with what type of code?

- HUD
- CUF
- BAM
- LAD

Manufactured housing is completely _________.

- factory assembled
- manually assembled
- built on site

An _________ is used to transport mobile homes to their site.

- integral chassis
- peripheral chassis
- secondary casing
- subsidiary casing

In recent years, factory-built housing has captured more than ______ percent of new housing in the US.

- 20
- 50
- 75
- 2

**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 a couple major model building codes in the United States that are comprehensive; that is, they cover all types of buildings and occupancies. The two major comprehensive building codes follow:
• International Building Code (IBC)
• International Residential Code for One- and Two-Family Dwellings (IRC)

You can read these codes at http://publicecodes.cyberregs.com/icod/.

FIGURE 1.7 Use of Model Building Codes in the United States

Visit http://www.iccsafe.org/gr/Pages/adoptions.aspx for the latest on building code adoptions around the United States.

Model building codes do not provide detailed specifications for all building materials and products, but instead refer to established industry standards. Several 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.

Seasoned designers spend countless hours in careful study and application of building codes and selected standards that relate to their area of practice. More importantly, 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 experience in the field. One of the most valuable sources of practical experience is the successes and failures of past designs and construction practices, as presented later in this article.

Structural Design Basics Quiz Part II
Metal plate-connected wood trusses are examples of what?

- Engineered Wood Products and Components
- Cold-Formed Steel
- Insulating Concrete Forms
- Masonry

Engineered Wood Products are composed of assemblies with structural properties that are __________ than the sum of their components.

- similar to or better
- worse
- always better

Engineered Wood Products generally offer improved _______ stability, increased structural capability, more efficient use of lumber resources, and increased structural capability.

- dimensional
- foundational
- lateral
- horizontal

Cold-formed steel framing was previously known as:

- light-gauge steel framing
- heavy-gauge steel framing
- slight-formed steel framing
- freeze-formed steel framing

Cold-formed steel framing has been produced for many years for _______ design and construction market.

- commercial
- industrial
- residential
- manufactured

Cold-formed steel framing is currently used in exterior and interior ______ in new housing starts.

- walls
- door frames
- window frames

What does ICF stand for in construction?
• Insulating Concrete Form
• Interior Casing Frame
• Isolated Construction Framing
• Invasive Concrete Fixtures

Masonry Construction represents less than _____ percent of annual housing starts.

• 10
• 50
• 1
• 35

Role of the 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:

• Conventional design. Sometimes referred to as “non-engineered” construction, conventional design relies on standard practice as governed by prescriptive building code requirements for conventional residential buildings; 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 pre-construction 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;
• non-conventional 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, atria, and other special features.
Housing Structural Performance

There are well over 130 million housing units in the United States, and more than half are single-family dwellings. Each year, at least 1 million new single-family homes and townhomes are constructed, along with thousands of multi-family structures, most of which are low-rise apartments. Therefore, a small percentage 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 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.

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 is drawn from a published study of homeowner warranty insurance records. The data does 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 does, 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 warranty claims based on the frequency and cost of a claim.

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, and sticking windows and 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 non-consequential 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.

**TABLE 1.1 Top Five House Defects Based on Homeowner Warranty Claims**

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

**Structural Design Basics Quiz Part III**

What are the two major building codes in the United States?

- IBC and IRC
• IBC and AFC
• NAB and IRC
• IRC and AFC

Building codes do not provide detailed specifications for all building materials and products, instead they refer to established ____________

• industry standards
• professionals with expertise
• habits in the profession
• company standards

Conventional design relies on ____________ for conventional residential buildings, while some parts of the structure may be specially designed.

• standard practice and building code requirements
• prefabricated construction and installment
• mass production of manufactured elements

More than half of the housing units in the United States are:

• single-family dwellings
• multi-family units
• affordable housing units

Some performance problems such as ____________ may not be realized for several years.

• foundation problems and subsurface soil conditions
• improperly designed windows and doors
• appliance failures

Code requirements must maintain a reasonable balance between ____________ and ____________.

• affordability, safety
• affordability, feasibility
• feasibility, safety
• safety, maintainability

What is the most common performance problem found in warranty claims on houses in the US?

• Defects in drywall installation
• Septic drain field
• Foundation walls
• Window frames
What is the most costly claim for a home-related performance problem?

- Foundation walls
- Trim and Moldings
- Window Frame
- Defects in drywall installation

**Housing Performance**

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. 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 be understood objectively as a prerequisite to guiding rational design and construction decisions. Proper design that takes into account the wind and earthquake loads and the structural analysis procedures 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 a 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.

Hurricane Andrew struck a densely populated area of south Florida on August 24, 1992, with the peak recorded wind speed exceeding 175 mph. 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 (see Figure 1.8). Given the distance between the shoreline and the housing stock, most damage resulted from wind, rain, and wind-borne debris, and not from
the storm surge. Table 1.2 summarizes the key construction characteristics of the homes that experienced Hurricane Andrew's highest winds. 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**

<table>
<thead>
<tr>
<th>Component</th>
<th>Construction Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of stories</td>
<td>80% one</td>
</tr>
<tr>
<td>Roof construction</td>
<td>81% gable</td>
</tr>
<tr>
<td>Wall construction</td>
<td>96% masonry</td>
</tr>
<tr>
<td>Foundation type</td>
<td>100% slab</td>
</tr>
<tr>
<td>Siding material</td>
<td>94% stucco</td>
</tr>
<tr>
<td>Roofing material</td>
<td>73% composition shingle</td>
</tr>
<tr>
<td>Interior finish</td>
<td>Primarily gypsum board</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Construction Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>18% two</td>
</tr>
<tr>
<td>13% hip</td>
</tr>
<tr>
<td>4% wood framed</td>
</tr>
<tr>
<td>18% tile</td>
</tr>
<tr>
<td>9% other</td>
</tr>
</tbody>
</table>

**FIGURE 1.8 Maximum Gust Wind Speeds Experienced in Hurricane Andrew**

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

<table>
<thead>
<tr>
<th>Component</th>
<th>Damage Frequency (percent of sampled homes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof sheathing</td>
<td>24% (64%)</td>
</tr>
<tr>
<td>Walls</td>
<td>2%</td>
</tr>
<tr>
<td>Foundation</td>
<td>0%</td>
</tr>
<tr>
<td>Roofing</td>
<td>77%</td>
</tr>
<tr>
<td>Interior finish</td>
<td>85%</td>
</tr>
</tbody>
</table>

<p>| |</p>
<table>
<thead>
<tr>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

21
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 prevented 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:

- While Hurricane Andrew exacted notable damage, overall residential performance was within expectations, given the magnitude of the event and the minimum code-required roof sheathing attachment (a 6d nail) relative to the south Florida wind climate.
- 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 greater damage than one-story homes (95 percent confidence level).
- Hip roofs experienced significantly less damage than gable roofs on homes with otherwise similar characteristics (95 percent confidence level).

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.

**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. Again, roofing (i.e., shingles) was the most common source of damage, occurring in 4 percent of the sampled housing stock. 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 having relatively few trees (wind exposure B). Hurricane Andrew denuded any trees in the path of the strongest winds. Clearly, housing performance in protected, non-coastal 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.

The Northridge Earthquake near Los Angeles, California, 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. 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.

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 wallboard provided lateral racking resistance. Most of the crawlspace foundations used full-height concrete or masonry stem walls, and 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

<table>
<thead>
<tr>
<th>Component</th>
<th>Frequency of Construction Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of stories</td>
<td>79% one</td>
</tr>
<tr>
<td>Wall sheathing</td>
<td>80% none</td>
</tr>
<tr>
<td>Foundation type</td>
<td>68% crawl space</td>
</tr>
<tr>
<td>Exterior finish</td>
<td>50% stucco/mix</td>
</tr>
<tr>
<td>Interior finish</td>
<td>60% plaster board</td>
</tr>
<tr>
<td></td>
<td>18% two</td>
</tr>
<tr>
<td></td>
<td>7% plywood</td>
</tr>
<tr>
<td></td>
<td>34% slab</td>
</tr>
<tr>
<td></td>
<td>45% stucco only</td>
</tr>
<tr>
<td></td>
<td>6% other/unknown</td>
</tr>
<tr>
<td></td>
<td>3% other</td>
</tr>
<tr>
<td></td>
<td>13% unknown</td>
</tr>
<tr>
<td></td>
<td>8% other</td>
</tr>
</tbody>
</table>

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

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

<table>
<thead>
<tr>
<th>Estimated Damage within Survey Area</th>
<th>No Damage</th>
<th>Low Damage</th>
<th>Moderate Damage</th>
<th>High Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation</td>
<td>90.2%</td>
<td>8.0%</td>
<td>0.9%</td>
<td>0.9%</td>
</tr>
<tr>
<td>Walls</td>
<td>98.1%</td>
<td>1.9%</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>Roof</td>
<td>99.4%</td>
<td>0.6%</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>Exterior finish</td>
<td>50.7%</td>
<td>46.1%</td>
<td>2.9%</td>
<td>0.3%</td>
</tr>
<tr>
<td>Interior finish</td>
<td>49.8%</td>
<td>46.0%</td>
<td>4.2%</td>
<td>0.0%</td>
</tr>
</tbody>
</table>

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 percentage of surveyed homes (about 2 percent) that experienced moderate to high foundation damage was 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 and lapboard 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 damage.
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 non-structural,
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.
Overall, the damage relative to key design features showed no discernible 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 the amount of damage and the 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; crawlspace 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, in and 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;
- using fully sheathed construction in high-hazard seismic regions;
- 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.

Summary

Housing in the U.S. 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 U.S. 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 over-design to compensate partially for real or perceived problems in installation quality.

Structural Design Basics Quiz Part IV

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 ____ construction.
• conventional
• unconventional
• commercial
• light-industrial
• ecofriendly
• atypical

Traditional ____ framing consists of closely spaced light vertical structural members that extend from the foundation sill to the roof plates.

• balloon
• platform
• continuous
• historical

____ framing is the modern adaptation of balloon framing whereby vertical members extend from the floor to the ceiling of each story.

• platform
• balloon
• continuous
• historical

Conventional or prescriptive construction practices are based as much on ______ as on technical analysis and theory.

• experience
• opinions
• law

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 ____ for standard framing lumber.

• 1.5 inches
• 1 inch
• 2.5 inches
• 350 mm
• 3 and ? inches

Wood truss roof framing _____ used in most new homes because it is generally less efficient than older stick-framing methods.

• is
• is not
An engineered wood product known as oriented strand board (OSB) is now substantially replacing __________.

- plywood
- balsa wood
- metal trusses
- steel work

_________ homes in the United States are site-built; that is, they follow a "stick framing" approach.

- many
- very few

T/F: An increasing number of today's site-built homes use components that are fabricated in an off-site plant.

- True
- False

____ 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
- Balloon
- Conventional
- Historical
- Straw bale

**Structural Design Concepts**

**Introduction**

This article 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, overall design approach, and how to inspect the structural design of a residential dwelling. With this conceptual background, it is hoped that the inspector will gain a greater appreciation for creative and efficient design of homes, particularly the many assumptions that must be made.
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. 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.

**Load Conditions & 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 the table, 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>
<thead>
<tr>
<th>Building Loads Categorized by Orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Vertical Loads</strong></td>
</tr>
<tr>
<td>Dead (gravity)</td>
</tr>
<tr>
<td>Live (gravity)</td>
</tr>
<tr>
<td>Snow (gravity)</td>
</tr>
<tr>
<td>Wind (uplift on roof)</td>
</tr>
<tr>
<td>Seismic and wind (overturning)</td>
</tr>
<tr>
<td>Seismic (vertical ground motion)</td>
</tr>
</tbody>
</table>

**Vertical Loads**

Gravity loads act in the same direction as gravity (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 (the weight of the construction) and any applied loads (the live load). For example, the tributary gravity load on a floor joist would include the uniform floor load (dead and live loads) 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 (or reactions), internal stresses (such as bending stresses, shear stresses, and axial stresses), and stability of the structural member or system. 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 behaviors. 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 (such as the 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 with proximity to changes in geometry (such as at the eaves, corners and ridges). Even though the wind loads are dynamic and highly variable, the design approach is based on a maximum static load or 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 that 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.

**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 or lateral load depends on the magnitude of the ground motion, the building’s mass, and the dynamic structural response characteristics (such as 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 or 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 to keep 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 \[\text{offset the overturning and uplift forces, as is often the case in lower design wind conditions and in many seismic design conditions.}\]

**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, according to the National Bureau of Standards (NBS). 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. But this document is narrowly scoped to address residential construction design.

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 (including 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 component than 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 a non-conservative solution, whereas the typical approach of only considering the floor joist member without composite system effect will lead to a conservative design.

The information presented here 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 non-load-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. In other words, 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, including the amount of openings and other factors.

This example of composite action due to the interaction of separate structural systems or sub-assemblies points to the improved structural response of the floor system such that it is able to carry more dead and live loads than if the partition wall were absent. One whole-house assembly test performed in 1965 demonstrated this effect. Hence, a double joist should not be required under a typical non-load-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, the designer should be aware of the concept when making design assumptions regarding light-frame residential construction.

At this point, the inspector should consider that the response of a structural system, and 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 (such as single members with standard tributary areas and assumed elastic behavior) and simplistic assumptions regarding load path can be substantial.
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.

Furthermore, because interior non-load-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.

Vertical Load Path

Figures 1 and Figure 2 below 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). 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. It may be feasible to combine these vertical forces and design a simple load path to accommodate wind uplift and overturning forces simultaneously.
Figure 1. Illustration of the Vertical Load Path for Gravity Loads
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 this 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 1), rather than an individual element single-handedly resisting the entire load originating from above. Thus, a framing system around a wall opening, and not just a header, comprises a load path.
Figure 3. Illustration of Wall and Window Framing Components

Figure 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 over-designed 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 lifetime 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, such as residential.

Consider the soil-bearing reaction at the bottom of the footing in Figure 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 lifetime 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% of the design live load, according to Chalk and Corotis. 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), 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, such as when it is furnished and/or occupied. In addition, other non-engineered 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 over-estimate 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 (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, and 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. 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.

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; AF&PA). 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; ICBO).

**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 3, the lateral load path in wood-framed construction involves entire structural assemblies (including 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 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 non-linearity of the load-displacement characteristics of wood-framed assemblies and their interconnections. Figure 4 below illustrates a deformed light-frame building under lateral load; the deformations are exaggerated for conceptual purposes.
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 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 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 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.

**Summary**

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 information serves as a resource for both inspectors and designers who are considering the installation and use of improved analytic methods when current analytic approaches may be lacking.

**Structural Design Concept Quiz Part I**

The goals of the structural design are generally defined by ________ and reflect the collective interpretation of the general public welfare.

- the law
- the blueprints
- the construction company

What type of load is a Wind load?

- Horizontal
- Vertical
- Diagonal

Gravity loads act in the ________ direction as gravity.
Gravity loads are generally ______ in nature.

- static
- dynamic
- mobile

Gravity loads are usually considered a ______ distributed or concentrated load.

- uniformly
- variably
- fluctuating

The ______ area is the area of the building construction that is supported by a structural element, including the dead load and any applied loads.

- tributary
- channel
- vertical

Wind uplift forces are generated by ______ pressures acting in an outward direction from the surface of the roof.

- negative
- positive
- neutral

Vertical forces are created by overturning reactions due to wind and ______ lateral loads acting on the overall building.

- seismic
- tributary
- electrical
- live

Which two types of loads apply to the entire building?

- Wind and Seismic Lateral
- Earth and Flood
- Gravity and Live
- Vertical and Flood
Lateral forces of wind are generated by positive wind pressures on the _______ face of the building and by negative pressures on the _______ face of the building.

- windward, leeward
- leeward, windward
- northward, windward
- windward, northward

Seismic lateral forces are generated by a structure’s dynamic inertia in response to _______ ground movement.

- cyclical
- linear
- perpendicular
- parallel

What type of load is generally minimized by elevating the structure on a properly designed foundation?

- Flood
- Wind
- Seismic
- Live

Lateral loads produce an _______ moment that must be offset by the dead load and connections of the building.

- overturning
- retracting
- nullifying

Load-sharing is found in _______ member systems and reflects the ability of the load on one member to be shared by another.

- repetitive
- constant
- simultaneous

The path through which loads are transferred is known as the:

- load path
- current path
- load circuit
- current circuit
is capable of transferring the loads that are realized throughout the structure from the point of load origination down to the foundation.

- A continuous load path
- A complex load path
- A temporary load path

Wind uplift load originates on the roof from suction forces that act _______ to the exterior surface of the roof.

- perpendicular
- parallel
- collateral

The load path itself can be considered ________.

- complex
- simple
- straightforward
- uncomplicated

It is common to consider only _______ of the dead load when analyzing a structure’s net wind uplift forces.

- two-thirds
- one-third
- one-half
- three-quarters

One benefit of traditional platform-framed home construction is that the floor and roof gravity loads are typically transferred through _______ points, not connections.

- bearing
- association
- interconnection

Connections may contribute little to the structural performance of homes with respect to _______ loads associated with gravity.

- vertical
- horizontal
- lateral

Structural Design Concepts Quiz Part II
T/F: Structural design involves the analysis of a proposed structure to show that its resistance or strength will meet or exceed a reasonable expectation.

- True
- False

A useful life of a structure implies considerations for all of the following time-varying risks, except for ____.

- occupant age
- corrosive environments
- occupant loads
- snow loads
- wind loads
- seismic loads

The balance between the two competing considerations of ____ help guides to determine the value in a building design and construction.

- performance and cost
- location and climate
- age and timing
- color and comfort

Because value is subjective, ____ 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.

- political
- social
- personal
- mathematical

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.

- True
- False

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 loads
- snow loads and human loads
- big loads and small loads
• diagonal forces and vectors

Gravity is considered a ____ load.

• vertical
• horizontal

Live loads are considered a ____ load.

• vertical
• horizontal

Soil with active lateral pressure is considered a ____ load.

• horizontal
• vertical

The following are all vertical loads, except for ____.

• wind
• snow
• dead
• seismic and wind (overturning)
• seismic (vertical ground motion)

The following are all horizontal loads, except for ____.

• live
• wind
• seismic (horizontal ground motion)
• flood (static and dynamic hydraulic forces)
• soil (active lateral pressure)

T/F: The tributary gravity load on a floor joist would include the uniform floor load (dead and live loads) applied to the area of floor supported by the individual joist.

• True
• False

____ 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.

• Wind
• Gravity
• Plumbing
Soil

Lateral forces from wind are generated by ____ wind pressures on the windward face of the building and by ____ pressures on the leeward face of the building, creating a combined push-and-pull effect.

- positive... negative
- negative... positive
- positive... positive
- negative... negative

**Structural Design Loads**

**General Information**

Loads are a primary consideration in any building design because they define the nature and magnitude of hazards and 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 (performance versus economy) of the finished product, it is essential to apply design loads realistically.

While the buildings we are considering in this article 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 here are based on applicable provisions of the ASCE 7 standard, Minimum Design Loads for Buildings and Other Structures. The standard represents an acceptable practice for building loads in the United States and is recognized in 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 article partly focuses on technical resources that are particularly relevant to the determination of design loads for residential structures. 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 practices, and the applicable building code relative to design loads as presented in this article, even though the variations may be considered technically sound.
The complete design of a home typically requires the evaluation of several different types of materials. Some material specifications use the allowable stress design or ASD approach, while others use load and resistance factor design or LRFD. Therefore, for a single project, it may be necessary to determine loads in accordance with both design formats. This article 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.

**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 for the designer, who should exercise discretion in any particular application.

The load combinations in Table 3.1 are simplified and tailored to specific applications in residential construction and the design of typical components and systems in a home. These and similar load combinations are often used in practice as shortcuts to those load combinations that govern the design result. This article makes effective use of the shortcuts and provides examples later in the article. The shortcuts are intended only for the design of residential light-frame construction.

**TABLE 3.1 Typical Load Combinations Used for the Design of Component or Systems**

<table>
<thead>
<tr>
<th>Component or System</th>
<th>ASD Load Combinations</th>
<th>LRFD Load Combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundations wall (gravity and soil lateral loads)</td>
<td>D = H</td>
<td>1.2D = 1.6H</td>
</tr>
<tr>
<td></td>
<td>D = H + L^2 + 0.3(L_w + S)</td>
<td>1.2D = 1.6H + 1.6L^2 + 0.5(L_w + S)</td>
</tr>
<tr>
<td>Headers, girders, posts, interior load-bearing walls and columns, footings (gravity loads)</td>
<td>D = L^2 + 0.3(L_w + S)</td>
<td>1.2D = 1.6L^2 + 0.5(L_w + S)</td>
</tr>
<tr>
<td></td>
<td>D = (L_w or S) + 0.3L^2</td>
<td>1.2D = 1.6(L_w or S) + 0.5L^2</td>
</tr>
<tr>
<td>Exterior load-bearing walls and columns (gravity and transverse lateral loads)</td>
<td>Same as immediately above plus</td>
<td>Same as immediately above plus</td>
</tr>
<tr>
<td></td>
<td>D = W</td>
<td>1.2D = 1.5W</td>
</tr>
<tr>
<td></td>
<td>D = 0.7E</td>
<td>1.2D = 1.0E / 0.7E</td>
</tr>
<tr>
<td></td>
<td>0.6L_w + W</td>
<td>1.2D = 1.5W</td>
</tr>
<tr>
<td>Roof rafters, trusses, and beams; roof and wall sheathing (gravity and wind loads)</td>
<td>0.6D + W</td>
<td>0.9D = (1.5W or 1.0E)</td>
</tr>
<tr>
<td></td>
<td>D = W^*</td>
<td>1.2D = 1.5W^*</td>
</tr>
<tr>
<td>Floor diaphragms and shear walls (in-plane lateral and overturning loads)</td>
<td>0.6D + (W or 0.7E)</td>
<td>0.9D + (1.5W or 1.0E)</td>
</tr>
</tbody>
</table>
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 section.

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. Furthermore, 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 (suction) pressures on the roof. Wind uplift loads must be resisted by continuous load path connections to the foundation or until offset by 0.6\( D \).

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.

**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.

### TABLE 3.2 Dead Loads for Common Residential Construction

**Notes:**

1. Reference unit conversions.
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

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminum</td>
<td>170</td>
</tr>
<tr>
<td>Copper</td>
<td>556</td>
</tr>
<tr>
<td>Steel</td>
<td>492</td>
</tr>
<tr>
<td>Concrete (normal weight with light reinforcement)</td>
<td>145-150 pcf</td>
</tr>
<tr>
<td>Masonry, brick</td>
<td>100-180 pcf</td>
</tr>
<tr>
<td>Masonry, concrete</td>
<td>85-135 pcf</td>
</tr>
<tr>
<td>Glass</td>
<td>160</td>
</tr>
<tr>
<td>Wood (approximately 10 percent moisture content)</td>
<td>29 pcf</td>
</tr>
<tr>
<td>- spruce pine (G = 0.42)</td>
<td>29 pcf</td>
</tr>
<tr>
<td>- spruce fir, rough (G = 0.34)</td>
<td>25 pcf</td>
</tr>
<tr>
<td>- southern yellow pine (G = 0.55)</td>
<td>38 pcf</td>
</tr>
<tr>
<td>- Douglas fir, heart (G = 0.5)</td>
<td>34 pcf</td>
</tr>
<tr>
<td>- hickory (G = 0.43)</td>
<td>30 pcf</td>
</tr>
<tr>
<td>- mixed oak (G = 0.68)</td>
<td>47 pcf</td>
</tr>
<tr>
<td>Water</td>
<td>62.4</td>
</tr>
<tr>
<td>Structural wood panels</td>
<td>36 pcf</td>
</tr>
<tr>
<td>- plywood</td>
<td>36 pcf</td>
</tr>
<tr>
<td>- oriented strand board</td>
<td>36 pcf</td>
</tr>
<tr>
<td>Gypsum board</td>
<td>48 pcf</td>
</tr>
<tr>
<td>Stone</td>
<td>96 pcf</td>
</tr>
<tr>
<td>- Granite</td>
<td>82 pcf</td>
</tr>
<tr>
<td>Sand, dry</td>
<td>96 pcf</td>
</tr>
<tr>
<td>Gravel, dry</td>
<td>106 pcf</td>
</tr>
</tbody>
</table>

### Live Loads

Live loads are produced by the use and occupancy of a building. Loads include those from human occupants, furnishings, non-fixed equipment, storage, and construction and maintenance activities. Table 3.4 provides recommended design live loads for residential buildings. As required to adequately define the loading condition, loads are presented in terms of uniform area loads (psf), concentrated loads (lbs), and uniform live 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**

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

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. 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.
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. 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.

**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 Ka to account for the fact that the soil is not actually fluid and that the pressure distribution is not necessarily triangular. The coefficient Ka 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 Foundation Basement 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**

\[
H = \frac{1}{2} q h (h) = \frac{1}{2} qh^2
\]

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), 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. 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.

**TABLE 3.5 Values of Ka, Soil Unit Weight, and Equivalent Fluid Density by Soil Type 1, 2, 3**
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.

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 (non-expansive) 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% 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.
Wind Loads

Wind produces non-static 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. 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.

Determination of Wind Loads

The following method for the design of residential buildings is based on a simplification of the ASCE 7 wind provisions; 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 for maps with greater detail), select a design wind speed for the site. The wind speed map in ASCE 7 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**

<table>
<thead>
<tr>
<th>Fastest mile (mph)</th>
<th>70</th>
<th>75</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
<th>120</th>
<th>130</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak gust (mph)</td>
<td>85</td>
<td>90</td>
<td>100</td>
<td>110</td>
<td>120</td>
<td>130</td>
<td>140</td>
<td>150</td>
</tr>
</tbody>
</table>

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.
Step 2

Make 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 article are derived from ASCE 7 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 non-coastal 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 10 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% (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 non-hurricane-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 non-hurricane-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. The lateral wind load must be determined for the two orthogonal directions on the building (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.

<table>
<thead>
<tr>
<th>Application</th>
<th>Lateral Pressure Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Vertical Projected Area (by slope)</td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td>0.0</td>
</tr>
<tr>
<td>3:12</td>
<td>0.3</td>
</tr>
<tr>
<td>6:12</td>
<td>0.5</td>
</tr>
<tr>
<td>9:12</td>
<td>0.8</td>
</tr>
<tr>
<td>Wall Projected Area</td>
<td>1.2</td>
</tr>
</tbody>
</table>

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 GCp 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.

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)

<table>
<thead>
<tr>
<th>Application</th>
<th>Pressure Coefficients (GCP)²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>-0.9, +0.4</td>
</tr>
<tr>
<td>Trusses, roof beams, ridge and hip/valley rafters</td>
<td>-1.2, +0.7</td>
</tr>
<tr>
<td>Rafters and truss panel members</td>
<td>-2.2, +1.0</td>
</tr>
<tr>
<td>Roof sheathing</td>
<td>-1.2, +1.0</td>
</tr>
<tr>
<td>Skylights and glazing</td>
<td></td>
</tr>
<tr>
<td>Roof uplift¹</td>
<td></td>
</tr>
<tr>
<td>- hip roof with slope between 3:12 and 6:12</td>
<td>-0.9</td>
</tr>
<tr>
<td>- hip roof with slope greater than 6:12</td>
<td>-0.8</td>
</tr>
<tr>
<td>- all other roof types and slopes</td>
<td>-1.0</td>
</tr>
<tr>
<td>Windward overhang²</td>
<td>+0.8</td>
</tr>
<tr>
<td>Wall</td>
<td>-1.2, +1.1</td>
</tr>
<tr>
<td>All framing members</td>
<td>-1.3, +1.2</td>
</tr>
<tr>
<td>Wall sheathing</td>
<td></td>
</tr>
<tr>
<td>Windows, doors, and glazing</td>
<td>-1.3, +1.2</td>
</tr>
<tr>
<td>Garage doors</td>
<td>-1.1, +1.0</td>
</tr>
<tr>
<td>Air-permeable claddings⁵</td>
<td>-0.9, 0.8</td>
</tr>
</tbody>
</table>

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.

Special Considerations

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.

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. Furthermore, 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

<table>
<thead>
<tr>
<th>Description</th>
<th>Velocity</th>
<th>Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 gram steel balls</td>
<td>130 fps</td>
<td>10 ft-lb</td>
</tr>
<tr>
<td>4.5-lb 2x4</td>
<td>40 fps</td>
<td>100 ft-lb</td>
</tr>
<tr>
<td>9.0-lb 2x4</td>
<td>50 fps</td>
<td>350 ft-lb</td>
</tr>
</tbody>
</table>

**Notes:**


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 1 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 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 under-designed 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.

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 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 inspection, maintenance and repair.

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- and three-story buildings.
- On average, hip roofs have demonstrated better performance than gable-end roofs.
- Moderate roof slopes (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.

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% 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 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)
Earthquake Loads

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. It is also similar to a simplified approach found in more recent building code development (ICC).

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); refer to Figure 3.4.

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.
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)

Map from American Society of Civil Engineers, ASCE

http://publicecodes.cyberregs.com/icod/ibc/index.htm

**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**

\[ V = \frac{1.2 \cdot S_{D5}}{R} \cdot W, \]

where,

- \( S_{D5} \) = 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. 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. 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 SDS 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, non-destructive 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.

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

**Equation 3.8-2**

\[
S_{DS} = \frac{2}{3}S_s(F_a)
\]

The value of Ss 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 Ss 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 Ss 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 Fa associated with a standard “firm” soil condition used for the design of residential buildings. Fa 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 Fa should be considered. Nonetheless, as noted in the Anchorage Earthquake of 1964 and again 30 years later in the Northridge Earthquake, 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)**

<table>
<thead>
<tr>
<th>$S_a$</th>
<th>0.25g</th>
<th>0.5g</th>
<th>0.75g</th>
<th>1.0g</th>
<th>1.25g</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_a$</td>
<td>1.6</td>
<td>1.4</td>
<td>1.2</td>
<td>1.1</td>
<td>1.0</td>
</tr>
</tbody>
</table>

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. 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**

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Seismic Response Modifier, $R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light-frame shear walls with wood structural panels used as bearing walls</td>
<td>6.0²</td>
</tr>
<tr>
<td>Light-frame shear walls with wall board/lath and plaster</td>
<td>2.0</td>
</tr>
<tr>
<td>Reinforced concrete shear walls</td>
<td>4.5</td>
</tr>
<tr>
<td>Reinforced masonry shear walls</td>
<td>3.5</td>
</tr>
<tr>
<td>Plain concrete shear walls</td>
<td>1.5</td>
</tr>
<tr>
<td>Plain masonry shear walls</td>
<td>1.25</td>
</tr>
</tbody>
</table>

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.

**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, the approach used in this article 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. 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.

**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.

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 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. It is now included in some of the latest seismic design provisions. Since it appears that redundancy factors have little technical basis and insufficient verification relative to light-frame structures, they are not explicitly addressed in this article. 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. 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 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 non-uniform 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 by redesigning the sill plate connection to accommodate the hold-down deformation and improve load distribution. As a non-structural 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. 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. 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.

**Other Load Conditions**

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. 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 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 loads noted above by using sound design detailing. For example, frost heave can be avoided by placing footings below a “safe” frost depth, building on non-frost-susceptible materials, or using other frost-protection methods. 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, D.C.

As noted at the beginning of this article, this article 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.
Structural Design Loads Quiz

T/F: Load combinations provide the basic set of building load conditions that establish the proportioning of multiple transient loads that may assume point-in-time values when the load of interest attains its extreme design value.

- True
- False

____ loads consist of the permanent construction material loads comprising the roof, floor, wall, and foundation systems, including claddings, finishes, and fixed equipment.

- Dead
- Live
- System
- Wind
- Framing

____ loads are produced by the use and occupancy of a building.

- Live
- Dead
- Conditional
- Determinant

As required to adequately define the loading condition, loads are presented in terms of uniform area loads (____), concentrated loads (____), and uniform live loads (____).

- psf? lbs? plf
- lbs? psf? plf
- kg? lbs? mm

T/F: 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, because theoretical analyses are usually based on homogeneous materials that demonstrate consistent compaction and behavioral properties.

- True
- False

Wind produces ____ on a structure at highly variable magnitudes.

- non-static loads
- static loads
T/F: 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.

- True
- False

T/F: 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.

- True
- False

Roof overhangs ____ uplift loads on roof tie-downs and the framing members that support the overhangs.

- increase
- decrease

T/F: One-story buildings are much more vulnerable to wind damage than two- and three-story buildings.

- False
- True

**Structural Design of Foundations**

**General Information**

A crawlspace is a building foundation that uses a perimeter foundation wall to create an under-floor space that is not habitable; the interior crawlspace 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.

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 floodwater 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 crawlspace 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 section covers:

- 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 (American Concrete Institute)-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. In addition, the designer is referred to the applicable design standards for symbol definitions and additional guidance, since the intent of this article is to provide supplemental instruction in the efficient design of residential foundations.

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. A brief discussion of the properties of concrete and masonry follows.

Concrete

The concrete compressive strength 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% 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 include:

- water reducers to improve the workability of concrete without reducing its strength;
- retarders used in hot weather to allow more time for placing and finishing concrete. Retarders may also reduce the early strength of concrete;
• accelerators to reduce the setting time, allowing less time for placing and finishing concrete. Accelerators may also increase the early strength of concrete; and
• air-entrainers used for concrete that will be exposed to freeze-thaw conditions and de-icing 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 of the steel (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 be 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 crowded 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 areas for reinforced concrete and masonry design.

<table>
<thead>
<tr>
<th>Size</th>
<th>Diameter (inches)</th>
<th>Area (square inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 3</td>
<td>3/8</td>
<td>0.11</td>
</tr>
<tr>
<td>No. 4</td>
<td>1/2</td>
<td>0.20</td>
</tr>
<tr>
<td>No. 5</td>
<td>5/8</td>
<td>0.31</td>
</tr>
<tr>
<td>No. 6</td>
<td>3/4</td>
<td>0.44</td>
</tr>
<tr>
<td>No. 7</td>
<td>7/8</td>
<td>0.60</td>
</tr>
<tr>
<td>No. 8</td>
<td>1</td>
<td>0.79</td>
</tr>
</tbody>
</table>

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’s compressive strength 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 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 non-moisture-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 parging 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%, 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% 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% 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's 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.

**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.
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 1-foot penetration (or 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 non-continuous or independent spread footings supporting columns or piers (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

<table>
<thead>
<tr>
<th>Presumptive Load-Bearing Value (psf)</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,500</td>
<td>Clay, sandy clay, silty clay, clayey silt, silt, and sandy silt</td>
</tr>
<tr>
<td>2,000</td>
<td>Sand, silty sand, clayey sand, silty gravel, and clayey gravel</td>
</tr>
<tr>
<td>3,000</td>
<td>Gravel and sandy gravel</td>
</tr>
<tr>
<td>4,000</td>
<td>Sedimentary rock</td>
</tr>
<tr>
<td>12,000</td>
<td>Crystalline bedrock</td>
</tr>
</tbody>
</table>
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 by using allowable stress design (ASD) load combinations.

**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.

In the next section, we’ll learn about design methods for concrete and gravel footings.

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.

**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 (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.

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, 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.

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. 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.

FIGURE 4.2 Critical Failure Planes in Continuous or Square Concrete
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 of the area described above.

FIGURE 4.2 Critical Failure Planes in Continuous or Square Concrete
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.

FIGURE 4.2 Critical Failure Planes in Continuous or Square Concrete
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.

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.

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 (e.g., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. In addition, the terms 4/3 $f'c$ and 4 $f'c$ are in units of pounds per square inch and represent “lower-bound” estimates of the ultimate shear stress capacity of unreinforced concrete.

\[
\phi V_s = \frac{V_{s,\text{basic}}}{\phi} \quad \text{factor of safety} \\
V_s, \phi V_s \quad \text{shear load (lb)} \\
\phi = 0.65 \quad \text{resistance factor}
\]

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 $f'c$ 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]

\[
\phi M_u = M_o \\
M_o = \frac{1}{8} q_s (b - t)^2 \\
q_s = \frac{P_o}{bd} \\
\phi F_u = \phi \sigma_d \sqrt{\frac{t}{S}} \\
S = \frac{1}{6} t^2 \\
\phi = 0.65
\]

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.

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 (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 (e.g., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. In addition, the terms $3\sqrt{f'c}$ 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 reinforced concrete.

**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 (e.g., 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 $\rho$ 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 $\phi M_n$ of the reinforced concrete. To aid the designer in shortcutting these calculations, design manuals provide design tables that correlate the nominal strength coefficient of resistance $R_n$ to the reinforcement ratio $\rho$ for a specific concrete compressive strength and steel yield strength.

**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.
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.

**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 non-moisture-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 a plain concrete wall. The following sections detail design methods for both wall types.

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 (reinforced or unreinforced) that may be appropriate for a given application. 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 (Lr or S)
- 1.2 D + 1.6 H + 1.6 (Lr 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 over-predict 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.

Plain Concrete Wall Design
ACI-318 allows the design of plain concrete walls with some limits, as discussed in ACI-318•220. 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 (minimize potential crack widening) by reasonable use of horizontal reinforcement.

ACI-318 limits plain concrete wall thickness to a minimum of 7-1/2 inches; however, the International One- 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 7-1/2-inch-minimum thickness requirement is obviously impractical for a short concrete stem wall, as in a crawlspace foundation.

Adequate strength needs to be provided and should be demonstrated by analysis in accordance with the ACI-318 design equations and the recommendations in this section. Depending on soil loads, analysis should confirm conventional residential foundation wall practice in typical conditions.

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 (perpendicular or out of plane) or parallel to the wall surface (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’s compressive strength. Alternatively, a wall can be reinforced.

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.
Figure 4.3 Variables Defined for Shear Calculations in a Plain Concrete Wall

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.

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. 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.

**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.

Reinforced concrete walls should be designed in accordance with ACI318•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 (perpendicular or out of plane) or parallel to the wall surface (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.
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 (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 “non-sway frame.” In concept, a sway frame is a frame (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 frame-like system. For more information on sway frames and their design procedure, refer to ACI318•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 non-sway 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 (non-pinned) of residential foundation walls.

\[
\frac{kd}{r} < 34 \quad \text{slenderness ratio}
\]

\[
r = \sqrt{\frac{I}{A}} = \sqrt{\frac{bd^3}{12}} = \sqrt{\frac{d^2}{12}} \quad \text{radius of gyration}
\]
Moment Magnifier Method

The moment magnifier method is an approximation method allowed in ACI318•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).

\[
\begin{align*}
\delta &= \frac{C_m}{1 - \frac{P_c}{\gamma \cdot 0.75 P_c}} \\
P_c &= \frac{\pi^2 E I}{(4 L)^2} \\
C_m &= 0.6 \\
or \\
C_m &= 1 	ext{ for members with transverse loads between supports} \\
M_{m, mm} &= P_c (0.6 + 0.03b) \\
E I &= \frac{0.5 E I_A}{\beta} \left( \frac{0.5 - \frac{A}{A_2}}{\beta} \right) \left( \frac{0.5 E I_A}{\beta} \right) \\
\beta &= 0.9 + 0.5 \frac{h}{L} - 1.2 \sqrt{r} \\
p &= \frac{A_1}{A_2} \\
\rho_2 &= \frac{P_{c, inst}}{P_c} \\
E_i &= 57,000 \sqrt{f_c} \text{ or } 0.5 \gamma_3 F_c 
\end{align*}
\]

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

<table>
<thead>
<tr>
<th>Minimum Wall Thickness (inches)</th>
<th>Maximum Wall Height (feet)</th>
<th>Factored Axial Load (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2,000</td>
<td>4,000</td>
</tr>
<tr>
<td>5.5</td>
<td>8</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1.12</td>
</tr>
<tr>
<td>7.5</td>
<td>8</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1.04</td>
</tr>
<tr>
<td>9.5</td>
<td>8</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1.00</td>
</tr>
</tbody>
</table>

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 (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% of the interaction curve.

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

![Interaction Diagrams for Plain and Reinforced Concrete Walls](image)

**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 (concrete beam) is required at the top of wall openings.

**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 $E_{CIg}$ for plain concrete walls and $E_{Cle}$ 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.

**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. 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 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.

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 (stirrups) of concrete lintels is not necessary for short spans (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**

![Diagram of the dimensions for bending and shear analysis of a concrete lintel](image)

**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.
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.

\[
M_k = \Phi M_E \\
M_k = \frac{wL^2}{12} \text{ for fixed-end beam model} \\
M_k = \frac{wL^2}{8} \text{ for simple span beam model} \\
\Phi M_k = \Phi A_{sf} \frac{d}{2} \\
a = \frac{A_{sf} f_y}{0.85 f_c b} \\
\Phi = 0.9
\]

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 Ec of the assumed concrete section; refer to ACI-318•9.5.2.3 to calculate the effective moment of inertia of the section.

Masonry Foundation Walls

Masonry foundation wall construction is common in residential construction. It is used in a variety of foundation types, including basements, crawlspaces, 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 as follows:

- \( D + H \)
- \( D + H + L + 0.3 \) (Lr or S)
- \( D + H + (Lr \text{ or } S) + 0.3 \) L

In light-frame homes, the first load combination typically governs masonry 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 over-predict the stresses in the wall. 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 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. 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 non-inspected 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 (block) are readily available in nominal 6-, 8-, 10- and 12-inch thicknesses. It is generally more economical if the masonry unit’s compressive strength 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.

**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).
- 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 non-expansive 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.

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 Foundation Walls**

<table>
<thead>
<tr>
<th>Nominal Wall Thickness</th>
<th>Maximum Unbalanced Backfill Height</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hollow Unit Masonry</td>
</tr>
<tr>
<td>6 inches</td>
<td>3</td>
</tr>
<tr>
<td>8 inches</td>
<td>5</td>
</tr>
<tr>
<td>10 inches</td>
<td>6</td>
</tr>
<tr>
<td>12 inches</td>
<td>7</td>
</tr>
</tbody>
</table>

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. 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.

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

<table>
<thead>
<tr>
<th>Type of Masonry Unit Construction</th>
<th>Mortar Type M or S</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Portland Cement/Lime (psi)</td>
</tr>
<tr>
<td>Normal to Red Joints</td>
<td></td>
</tr>
<tr>
<td>Solid</td>
<td>40</td>
</tr>
<tr>
<td>Hollow</td>
<td>25</td>
</tr>
<tr>
<td>Ungrouted</td>
<td>60</td>
</tr>
<tr>
<td>Fully grouted</td>
<td></td>
</tr>
<tr>
<td>Parallel to Red Joints in Running Bond</td>
<td></td>
</tr>
<tr>
<td>Solid</td>
<td>80</td>
</tr>
<tr>
<td>Hollow</td>
<td></td>
</tr>
<tr>
<td>Ungrouted/Partially grouted</td>
<td>50</td>
</tr>
<tr>
<td>Fully grouted</td>
<td>80</td>
</tr>
</tbody>
</table>

**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 (perpendicular or out of plane) and parallel to the wall surface (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.
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.

Combined Axial Compression and Flexural Capac

The following equations from ACI-530 determine the relationship of the combined effects of axial load and bending moment on a masonry wall.
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 (soil) loads and offset by axial loads (dead loads).

Even though an unreinforced masonry wall may calculate as adequate, the designer may consider adding a nominal amount of reinforcement to control cracking.

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.

Reinforced Masonry Design
When unreinforced concrete masonry wall construction does not satisfy all design criteria (e.g., 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. 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.

**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 (perpendicular or out of plane) and parallel to the wall surface (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.

\[
\begin{align*}
F_v &= F_v \\
F_v &= \frac{V}{bd} \\
F_r &= 1.0 \sqrt{f_m} \quad 50 \text{psi for flexural members} \\
F_v &= \frac{1}{3} 4 - \frac{M}{Vd} \sqrt{f_m} \quad 80 - 45 \frac{M}{Vd} \quad \text{psi for shear walls where } \frac{M}{Vd} < 1 \\
F_v &= 1.0 \sqrt{f_m} \quad 35 \text{psi for shear walls where } \frac{M}{Vd} \geq 1
\end{align*}
\]
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 (e.g., 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**
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 Capac**

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.

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.
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 (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.

\[ \Lambda_{x,\text{required}} = \frac{M}{f_y d} \]
\[ \Lambda_{y,\text{min}} = 0.0013bt \]
\[ \Lambda_{h,\text{min}} = 0.0007bt \]

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 (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.

**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**
Granular (gravel or crushed rock) footings are sized accordingly. 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 preservative-treated and rated for below-ground application.
- Drainage systems are selected in accordance with foundation type (e.g., basement or crawlspace) and soil type. Foundation wall moisture-proofing is also required (i.e., polyethylene sheathing).

### TABLE 4.7 Preservative-Treated Wood Foundation Framing

<table>
<thead>
<tr>
<th>Maximum Unbalanced Backfill Height (feet)</th>
<th>Nominal Stud Size</th>
<th>Stud Center-to-Center Spacing (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>2x6</td>
<td>16</td>
</tr>
<tr>
<td>6</td>
<td>2x6</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>2x8</td>
<td>12</td>
</tr>
</tbody>
</table>

- 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 sanded board or the basement floor to resist shear forces from the backfill.

**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; and
- **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. 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).

**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 ACI302•2.1. Where interior columns and load-bearing walls bear on the slab, the slab is typically thickened and may be nominally reinforced. 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
prevention of 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 (e.g., 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.

**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 non-engineered 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 (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, D.C., offer design guidance. FEMA is also in the process of updating the Coastal Construction Manual.

The habitable portion of buildings in coastal “A zones” (non-velocity 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 crawlspace 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:

\[ P_a = \frac{W_r h}{s F} \]

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 factor. 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 pre-drill 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.

**Frost Protection**

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

**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;
- non-frost-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% fines (as measured by a #200 sieve) are considered non-frost-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 (e.g., 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**

<table>
<thead>
<tr>
<th>Air-Freezing Index (F-Days)</th>
<th>Footing Depth (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250 or less</td>
<td>12</td>
</tr>
<tr>
<td>500</td>
<td>18</td>
</tr>
<tr>
<td>1,000</td>
<td>24</td>
</tr>
<tr>
<td>2,000</td>
<td>36</td>
</tr>
<tr>
<td>3,000</td>
<td>48</td>
</tr>
<tr>
<td>4,000</td>
<td>60</td>
</tr>
</tbody>
</table>

**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. 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 (e.g., 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. 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
Permafrost

Design of residential foundations on permafrost is beyond the scope of this article. 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.

Structural Design of Foundations Quiz

T/F: A foundation transfers the load of a structure to the earth and resists loads imposed by the earth.

- True
- False

In North America, the most common residential foundation materials are ____ and cast-in-place concrete.

- concrete block
- treated wood
- stone
- brick units

The concrete slab on grade is the most popular foundation type in the ____ of the United States.

- Southeast
- Northeast
- Northwest
- Midwest

Crawlspace are common in the Northwest.

- True
- False

Basement foundations are common in Florida.

- False
- True
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.

- Pile
- Basement
- Stem wall
- Slab

A ___ is a building foundation that uses a perimeter foundation wall to create an under-floor space that is not habitable; the interior crawlspace elevation may or may not be below the exterior finish grade.

- crawlspace
- slab
- monolithic slab
- post and pier

A ___ with an independent stem wall is a concrete floor supported by the soil independently of the rest of the building.

- slab on grade
- basement
- crawlspace
- pile and grade beam
- monolithic

T/F: Piles can be used to isolate a structure from expansive soil movements.

- True
- False

T/F: Post-and-pier foundations can provide an economical alternative to crawlspace perimeter wall construction.

- True
- False

The concrete compressive strength used in residential construction is typically either ___, although other values may be specified.

- 2,500 or 3,000 psi
- 500 or 750 psi
- 2,000 or 3,000 lbs
- 1,000 or 1,500 spf
Given that concrete strength increases at a diminishing rate with time, the specified compressive strength is usually associated with the strength attained after ____ of curing time, at which time, concrete generally attains about 85% of its fully cured compressive strength.

- 28 days
- 7 days
- 24 hours
- 2 years

T/F: Residential foundation walls are typically constructed with a general-purpose Portland cement used for the vast majority of construction projects.

- True
- False

T/F: 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.

- True
- False

____ is the measure of concrete consistency; the higher the slump, the wetter the concrete and the easier it flows.

- Slump
- Bump
- Hump
- Dump
- Pour Ratio Level (PRL)

Concrete has high ____ strength but low ____ strength; therefore, reinforcing steel is often embedded in the concrete to provide additional tensile strength and ductility.

- compressive? tensile
- tensile? compressive

T/F: 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.

- True
- False

Steel reinforcement is available in Grade 40 or Grade 60, and most reinforcement in the U.S. market today is Grade ____.
Concrete masonry units (CMU) are commonly referred to as_____, and they are composed of Portland cement, aggregate and water.

- concrete blocks
- masonry blocks
- red brick
- stoneware
- stone block

**Structural Design of Wood Framing**

**General Information**

**Structural Design of Wood Framing for the Home Inspector**

This article addresses elements of above-grade structural systems in residential construction. The residential construction material most commonly used above grade in North America is light-frame wood; therefore, we’ll focus 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 (shear walls and diaphragms) must be approached from a system design perspective. Connections 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.

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 the connection of the floor system to the foundation. 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 inspectors,
system-based design principles are addressed here.

The design of the above-grade structure involves the following structural systems and assemblies:

- floors;
- walls; and
- roofs.

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), although the load-resistance factored design (LRFD) method is now available as an alternative. The ASD method is detailed in the National Design Specification for Wood Construction (NDS) at http://www.awc.org/standards/nds.php and its supplement (NDS-S). The reader is encouraged to obtain the NDS commentary to develop a better understanding of the rationale and substantiation for the NDS.

Let’s look at the NDS equations in general, which includes design examples that detail the appropriate use of the equations for specific structural elements or systems in light, wood-
framed construction, focusing primarily on framing with traditional dimensional lumber, giving some consideration to common engineered wood products. Other wood framing methods, such as post-and-beam construction, are not explicitly addressed here, although much of the information is relevant. However, system considerations and system factors presented here are only relevant to light, wood-framed construction using dimensional lumber.

Regardless of the type of structural element, the inspector 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.

While prescriptive design tables or span tables and similar design aids commonly used in residential applications are not included herein, the inspector 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), contain prescriptive design and construction provisions for conventional residential construction. 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). The inspector 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.

Material Properties

It is essential that a residential inspector 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.

Lumber

As with all materials, the inspector must consider wood’s strengths and weaknesses. A comprehensive source of technical information on the characteristics of wood is the Wood Engineering Handbook, Second Edition (Forest Products Laboratory). 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 substance that, as a structural material, demonstrates unique and complex characteristics. Wood’s structural properties can be traced back to its natural composition. Wood is foremost a non-homogeneous, 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 the tree’s annual growth rings, which
determine the properties of the wood along three orientations: tangential, radial and longitudinal.

Given that lumber is cut from logs in a longitudinal direction, the grain is parallel to the length of the 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 softwoods (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 U.S.

**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 inspector 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.
- Dimensional 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 dimensional 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 inspector should consult a lumber supplier or contractor regarding locally available lumber species and grades.

Machine stress-rated (MSR) and machine-evaluated lumber (MEL) are subjected to non-destructive testing of each piece. The wood member is then marked with the appropriate grade stamp, which includes the allowable bending stress (Fb) 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 the American Softwood Lumber Voluntary Product Standard, which is maintained by the National Institute for Standards and Technology (NIST). 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%; its loss results in changes in both volume (i.e., shrinkage) and structural properties. The strength of wood peaks at about 10 to 15% MC.

Given that wood generally has an MC of more than 30% when cut and may dry to an equilibrium moisture content (EMC) of 8 to 10% in a 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% 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%. The tabulated design values in the NDS are based on a moisture content of 19% for dimensional lumber.

The inspector 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 inspector might have to account for differential shrinkage by isolating members that will shrink from those that will maintain dimensional stability. The inspector 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.

 Shrinking and swelling can be estimated 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 (parallel to the 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% 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).

Wood members that are in contact with the ground 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 and for exterior decks. It is important to specify the correct level of treatment: 0.4 pcf retention for non-ground-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 (mud tunnels) on the surface of foundation walls. Since the presence of termites lends itself to visual detection, wood-framed homes require periodic inspection for signs of termites.

**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 three to five. 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 inspector 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-foot 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 for industrial and construction plywood. OSB products are performance-rated. However, these standards are voluntary, and not all wood-based panel products are rated accordingly. The rating system of the APA-Engineered Wood Association (formerly the American Plywood Association) for structural wood panel sheathing products and those used by other structural panel trademarking organizations are based on the U.S. Department of Commerce's voluntary product standards.

The veneer grade of plywood is associated with the veneers used on the exposed faces of a panel as follows:

- Grade A: 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; and
- 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

**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, NDS includes the most up-to-date design values based on test results from an eight-year, full-scale testing program that used lumber samples from mills across the United States and Canada.

Characteristic structural properties for use in allowable stress design and load and resistance factor design are used to establish design values. 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 5th-percentile test value. The 5th percentile represents the value that 95% 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.

The reduction factors in Table 5.1 are derived as follows, as reported in ASTM D2915 (ASTM):

- \( F_b \) reduction factor = \(\frac{10}{16} \) load-duration factor \(\times\)\(\frac{10}{13} \) safety factor; 
- \( F_t \) reduction factor = \(\frac{10}{16} \) load-duration factor \(\times\)\(\frac{10}{13} \) safety factor; 
- \( F_v \) reduction factor = \(\frac{10}{16} \) load-duration factor \(\times\)\(\frac{4}{9} \) stress-concentration factor \(\times\)\(\frac{8}{9} \) safety factor; 
- \( F_c \) reduction factor = \(\frac{2}{3} \) load-duration factor \(\times\)\(\frac{4}{5} \) safety factor; and 
- \( F_c \perp \) reduction factor = \(\frac{2}{3} \) end-position factor.

**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.
### TABLE 5.1 Design Properties and Associated Reduction Factors for ASD

<table>
<thead>
<tr>
<th>Stress Property</th>
<th>Reduction Factor</th>
<th>Basis of Estimated Characteristic Value from Test Data</th>
<th>Limit State</th>
<th>ASTM Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extreme fiber stress in bending, ( F_{sb} )</td>
<td>( \frac{1}{2.1} )</td>
<td>Fifth percentile</td>
<td>Ultimate capacity</td>
<td>D1990</td>
</tr>
<tr>
<td>Tension parallel to grain, ( F_t )</td>
<td>( \frac{1}{2.1} )</td>
<td>Fifth percentile</td>
<td>Ultimate capacity</td>
<td>D1990</td>
</tr>
<tr>
<td>Shear parallel to grain, ( F_s )</td>
<td>( \frac{1}{4.1} )</td>
<td>Fifth percentile</td>
<td>Ultimate capacity</td>
<td>D245</td>
</tr>
<tr>
<td>Compression parallel to grain, ( F_c )</td>
<td>( \frac{1}{1.9} )</td>
<td>Fifth percentile</td>
<td>Ultimate capacity</td>
<td>D1990</td>
</tr>
<tr>
<td>Compression perpendicular to grain, ( F_{cp} )</td>
<td>( \frac{1}{1.5} )</td>
<td>Mean</td>
<td>0.04&quot; deflection(^1)</td>
<td>D245</td>
</tr>
<tr>
<td>Modulus of elasticity, ( E )</td>
<td>( \frac{1}{1.0} )</td>
<td>Mean</td>
<td>Proportional limit(^2)</td>
<td>D1990</td>
</tr>
</tbody>
</table>

### TABLE 5.2 Adjustment Factor Applicability to Design Values for Wood

<table>
<thead>
<tr>
<th>Design Properties(^1)</th>
<th>( C_p )</th>
<th>( C_r )</th>
<th>( C_R )</th>
<th>( C_f )</th>
<th>( C_P )</th>
<th>( C_B )</th>
<th>( C_D )</th>
<th>( C_Y )</th>
<th>( C_X )</th>
<th>( C_T )</th>
<th>( C_C )</th>
<th>( C_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_b )</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>( F_t )</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>( F_s )</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>( F_c )</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>( F_{cp} )</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>( E )</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

Key to adjustment factors:

- **CD**, Load Duration Factor, applies when loads are other than the normal 10-year duration.
- **Cr**, Repetitive Member Factor, applies to bending members in assemblies with multiple members spaced at maximum 24 inches on center.
- **CH**, Horizontal Shear Factor, applies to individual or multiple members with regard to horizontal, parallel-to-grain splitting.
- **CF**, Size Factor, applies to member sizes/grades other than standard test specimens, but does not apply to southern yellow pine.
- **CP**, Column Stability Factor, applies to lateral support condition of compression members.
- **CL**, Beam Stability Factor, applies to bending members not subject to continuous lateral support on the compression edge.
- **CM**, Wet Service Factor, applies where the moisture content is expected to exceed 19% for extended periods.
- **Cfu**, Flat Use Factor, applies where dimensional lumber 2 to 4 inches thick is subject to a bending load in its weak axis direction.
- **Cb**, Bearing Area Factor, applies to members with bearing less than 6 inches and not nearer than 3 inches from the members’ ends.
• CT, Buckling Stiffness Factor, applies only to maximum 2x4 dimensional lumber in the top chord of wood trusses that are subjected to combined flexure and axial compression.
• CV, Volume Factor, applies to Glulam® bending members loaded perpendicular to the wide face of the laminations in strong axis bending.
• Ct, Temperature Factor, applies where temperatures exceed 100° F for long periods; not normally required when wood members are subjected to intermittent higher temperatures, such as in roof structures.
• Ci, Incising Factor, applies where structural-sawn lumber is incised to increase penetration of preservatives with small incisions cut parallel to the grain.
• Cc, Curvature Factor, applies only to curved portions of glued, laminated bending members.
• Cf, Form Factor, applies where bending members are either round or square with diagonal loading.

Load Duration Factor (CD)

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 (CD) as shown in Table 5.3. Values of the load duration factor, CD, 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. CD 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, CD, is equal to 1.6.
TABLE 5.3 Recommended Load Duration Factors for ASD

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Load Duration</th>
<th>Recommended C&lt;sub&gt;D&lt;/sub&gt; Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent (dead load)</td>
<td>Lifetime</td>
<td>0.9</td>
</tr>
<tr>
<td>Normal</td>
<td>Ten years</td>
<td>1.0</td>
</tr>
<tr>
<td>Occupancy (live load)&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Ten years to seven days</td>
<td>1.0 to 1.25</td>
</tr>
<tr>
<td>Snow&lt;sup&gt;2&lt;/sup&gt;</td>
<td>One month to seven days</td>
<td>1.15 to 1.25</td>
</tr>
<tr>
<td>Temporary construction</td>
<td>Seven days</td>
<td>1.25</td>
</tr>
<tr>
<td>Wind and seismic&lt;sup&gt;3&lt;/sup&gt;</td>
<td>Ten minutes to one minute</td>
<td>1.6 to 1.8</td>
</tr>
<tr>
<td>Impact</td>
<td>One second</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Repetitive Member Factor (Cr)

When three or more parallel dimensional 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% 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, Fb.

TABLE 5.4 Recommended Repetitive Member Factors for Dimension Lumber Used in Framing Systems

<table>
<thead>
<tr>
<th>Application</th>
<th>Recommended C&lt;sub&gt;r&lt;/sub&gt; Value</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two adjacent members sharing load&lt;sup&gt;4&lt;/sup&gt;</td>
<td>1.1 to 1.2</td>
<td>AF&amp;PA,1996b</td>
</tr>
<tr>
<td>Three adjacent members sharing load&lt;sup&gt;4&lt;/sup&gt;</td>
<td>1.2 to 1.3</td>
<td>ASAE, 1997</td>
</tr>
<tr>
<td>Four or more adjacent members sharing load&lt;sup&gt;4&lt;/sup&gt;</td>
<td>1.3 to 1.4</td>
<td>ASAE, 1997</td>
</tr>
<tr>
<td>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.)&lt;sup&gt;4&lt;/sup&gt;</td>
<td>1.15</td>
<td>NDS</td>
</tr>
<tr>
<td>Wall framing (units) 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&lt;sup&gt;5&lt;/sup&gt;</td>
<td>1.5–2x4 or smaller</td>
<td>AF&amp;PA, 1996b, SBCCI, 1999 Polomchuk, 1975</td>
</tr>
</tbody>
</table>

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 inspector as an alternative method based on various sources of technical information, including certain standards, code recognized guidelines, and research studies.
**Horizontal Shear Factor (CH)**

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 CH of greater than 1 should typically apply when repetitive framing or built-up members are used. For members with no splits, CH equals 2.

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% 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, CH, of 2 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 CH factor applies only to the allowable horizontal shear stress, Fv. As an interim consideration regarding horizontal shear at notches and connections in members, a CH value of 1.5 is recommended for use with provisions in NDS•3.4.4 and 3.4.5 for dimensional lumber only.

**Size Factor (CF)**

Tabulated design values in the NDS-S are based on testing conducted on members of certain sizes. The specified depth for dimensional 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., CF=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 CF. 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.

**Column Stability Factor (CP)**

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, CP, adjusts the tabulated compression stresses to account for the possibility of column buckling. For rectangular or non-symmetric columns, CP must be determined for both the weak- and strong-axis bracing conditions. CP 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 CP factor applies to the allowable compressive stress parallel to grain, \( F_c \), as shown in Table 5.2.

**Beam Stability Factor (CL)**

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., the 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 inspector should modify the tabulated bending design values by using the beam stability factor, CL, to account for the possibility of lateral-torsional buckling. For glued laminated timber bending members, the volume factor (CV) and beam stability factor (CL) are not applied simultaneously; thus, the lesser of these factors applies. Refer to the NDS 3.3.3 for the equations used to calculate CL.

**Structural Evaluation**

As with any structural design, the inspector 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
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 centerline 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.

\[
\begin{align*}
  f_b &= F_b / A \quad \text{(basic design check for bending stress)} \\
  F_b &= E_f \times x \quad \text{(applicable adjustment factors per Section 5.2.4)} \\
  f_b &= M_c / S \quad \text{(extreme fiber bending stress due to bending moment from transverse load)} \\
  S &= \frac{1}{6} bd^3 \quad \text{(section modulus of rectangular member)} \\
  I &= \frac{1}{12} bd^4 \quad \text{(moment of inertia of rectangular member)} \\
  c &= \frac{d}{2} \quad \text{(distance from extreme fiber to neutral axis)}
\end{align*}
\]

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 textbooks. Because
dimensional lumber is solid and rectangular, the simple equation for $f_v$ is most commonly used.

\[ f_v = \frac{F_v}{A} \quad \text{basic design check for horizontal shear} \]

\[ F_v = F_v \times \quad \text{(applicable adjustment factors per Section 5.2.4)} \]

\[ f_v = \frac{VQ}{lb} \quad \text{horizontal shear stress (general equation)} \]

\[ f_v = \frac{3V}{2A} \text{ for maximum horizontal shear stress at the neutral axis of solid rectangular members} \]

**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 inspector 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.

\[ f_{c\perp} = \frac{F_{c\perp}}{A_b} \quad \text{stress perpendicular to grain due to load, P, on net bearing area, A_b.} \]

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 inspector 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 over-stressed 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 inspector calculates the components of the combined stress interaction equations that are given below and found in the NDS.

\[
\begin{align*}
\text{Combined bending and axial tension design check} & : \quad \frac{f_t}{F_t} + \frac{f_b}{F_b^*} \leq 1 \\
\text{Combined bending and axial compression design check} & : \quad \frac{f_c}{F_c} + \frac{f_{bl}}{F_{bl} \left(1 - \frac{f_c}{F_{de1}}\right)} + \frac{f_{b2}}{F_{b2} \left(1 - \frac{f_c}{F_{de2}}\right)} \leq 1
\end{align*}
\]

**Compression and Column Stability**

For framing members that support axial loads only (i.e., columns), the inspector 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 inspector 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 inspector performs the calculations necessary to determine the stability factor, CP, in accordance with NDS 3.7. When a column has continuous lateral support in two directions, buckling is not an issue and CP = 1.0. If, however, the column is free to buckle in one or more directions, CP must be evaluated for each direction of possible buckling. The evaluation must also consider the spacing of intermediate bracing, if any, in each direction.
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. 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. Tension stresses in wood members are checked by using the equations below in accordance with NDS 3.8.

\[
\begin{align*}
F_t & = F_{t_1} \times \text{basic design check for tension parallel to grain} \\
F_{t_1} & = F_t \times \text{(applicable adjustment factors per Section 5.2.4)} \\
F_t & = \frac{P}{A} \text{stress in tension parallel to grain due to axial tension load, } P, \text{ acting on the member's cross-sectional area, } A
\end{align*}
\]

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 inspector 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.

Structural Serviceability
Deflection Due to Bending

The NDS does not specifically limit deflection but rather defers to inspector 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.

\[
\Delta_{\text{estimate}} = \Delta_{\text{allow}} = \frac{f_{\text{load and span}}}{EI} \quad \text{(see Table 5.5 for value of denominator)}
\]

If a deflection check proves unacceptable, the inspector 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 inspector judgment. Table 5.5 provides recommended deflection limits. Certainly, if a modest adjustment to a deflection limit results in a more efficient design, the inspector 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% of the nominal design wind load.

<table>
<thead>
<tr>
<th>Table 5.5 Recommended Allowable Deflection Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Element or Condition</strong></td>
</tr>
<tr>
<td>Rafters without attached ceiling finish</td>
</tr>
<tr>
<td>Rafters with attached ceiling finishes and trusses</td>
</tr>
<tr>
<td>Ceiling joists with attached finishes</td>
</tr>
<tr>
<td>Roof girders and beams</td>
</tr>
<tr>
<td>Walls</td>
</tr>
<tr>
<td>Headers</td>
</tr>
<tr>
<td>Floors</td>
</tr>
<tr>
<td>Floor girders and beams</td>
</tr>
</tbody>
</table>

Given that system effects influence the stiffness of assemblies in a manner similar to that of bending capacity, 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% or more. In areas where partitions add to the rigidity of the supporting floor, deflection can be overestimated by more than 50% (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 \( /360 \), 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**

<table>
<thead>
<tr>
<th>Framing System</th>
<th>Multiply single member deflection estimate by:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light-wood-frame floor system with minimum 2x8 joists, minimum 3/4-inch-thick sheathing, and standard fastening</td>
<td>0.85—Uniform load</td>
</tr>
<tr>
<td>Light-wood-frame floor system as above, but with glued and nailed sheathing</td>
<td>0.75—Uniform load</td>
</tr>
<tr>
<td>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</td>
<td>0.7–2x4</td>
</tr>
<tr>
<td></td>
<td>0.8–2x6</td>
</tr>
</tbody>
</table>

**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 inspector wishing to limit vibration beyond that implied by the traditional use of an \( /360 \) deflection limit (FHA, 1958; Woeste and Dolan, 1998).

- For floor joist spans less than 15 feet, a deflection limit of \( /360 \) considering design live loads only may be used, where 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 /480 should be used for spans greater than 15 feet, where is the clear span of the member in inches.
• When calculating deflection based on the above rules of thumb, the inspector 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 /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 two 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% 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.

Shrinking and swelling 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.
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 multi-family buildings (refer to local building codes).

General Information

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 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.
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.

When inspecting any structural element, the inspector must first determine the loads acting on the element. 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:

- \( D + L \)
- \( D + L + 0.3 \) (Lr or S)
- \( D + (Lr \text{ or } S) + 0.3L \)

**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). Therefore, it is usually not necessary to design conventional floor joists for residential construction. To obtain greater economy or performance, however, inspectors 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.

For different joist applications, such as a continuous multiple span, the inspector should use the appropriate beam equations 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. 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**

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. Under such conditions, the maximum bending moment experienced by any single joist is reduced by more than 60%. A similar reduction in the shear loading (and end reaction) of the loaded joist also results, with exceptions 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 Cr factor of 1.5 or more. 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 dimensional 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 dimensional 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 dimensional lumber joists greater than 12 inches in depth.
According to the study, bridging may be considered necessary for 2x10 and 2x12 dimensional 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.

**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 dimensional 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 dimensional 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 inspector 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, CL, is determined in accordance with NDS•3.3.3; however, for girders supporting floor framing, lateral support is considered to be continuous and CL = 1.

**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.

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.

Site-fabricated beams include plywood box beams, plywood I-beams, and flitch plate beams. Plywood box beams are fabricated from continuous dimensional 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 dimensional 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 dimensional 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-inch-thick and about 1/4-inch less in depth than the dimensional 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 dimensional 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. The International One- and Two-Family Dwelling Code (ICC) provides a simple prescriptive table for plywood box beam headers.

**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. 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 inspector 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 assist in specifying plywood subflooring suitable for heavy concentrated loads from vehicle tire loading.

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. 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. 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. 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**

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Size and Type of Fastener</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood and wood structural panels, subfloor sheathing to framing</td>
<td>6d nail</td>
</tr>
<tr>
<td>1/2-inch and less</td>
<td>6d nail</td>
</tr>
<tr>
<td>19/32- to 1-inch</td>
<td>8d nail</td>
</tr>
<tr>
<td>1-1/8- to 1-1/4-inch</td>
<td>10d nail or 8d deformed shank nail</td>
</tr>
<tr>
<td>Plywood and wood structural panels, combination subfloor/underlay to framing</td>
<td>8d nail or 6d deformed shank nail</td>
</tr>
<tr>
<td>3/4-inch and less</td>
<td>8d nail or 6d deformed shank nail</td>
</tr>
<tr>
<td>7/8- to -inch</td>
<td>8d nail</td>
</tr>
<tr>
<td>1-1/8- to 1-1/4-inch</td>
<td>10d nail or 8d deformed shank nail</td>
</tr>
</tbody>
</table>

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.

**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 multi-family buildings.

**General Information**

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 dimensional 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.
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% 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. 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 dimensional 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.

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 inspector uses the system strength inherent in the construction.
When inspecting wall elements, the inspector needs to consider the load combinations, particularly the following ASD combinations of dead, live, snow and wind loads:

- \( D + L + 0.3 \ (L_r \ or \ S) \)
- \( D + (L_r \ or \ S) + 0.3 \ L \)
- \( D + W \)
- \( D + 0.7E + 0.5L + 0.2S \)

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 inspector 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.

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 inspector judgment, experience, and knowledge of the critical design conditions.

**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 non-load-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.

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 \((l_e/d)_x\). 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, CP, for a wall system rather than for a single column in accordance with NDS 3.7.1, the inspector must exercise judgment with respect to the calculation of the effective length, \(e\), 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 inspector 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 non-load-bearing. Non-load-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 abut. 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 wall-covering 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*.

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 or 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. 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 inspector should recommend load-bearing walls in accordance with the previous section on exterior load bearing walls.

**Non-Load-Bearing Partition**

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 non-load-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 earlier sections apply as appropriate.

Headers

Load-bearing headers are horizontal members that carry loads from a wall, ceiling, 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. This guide considers headers consisting of double members to be repetitive members; therefore, a repetitive member factor, Cr, of 1.1 to 1.2 should apply, along with a live load deflection limit of /240. Large openings or especially heavy loads may require stronger members, such as engineered wood beams, hot-rolled steel, or flitch plate beams.

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 testing determined whether an adjustment factor (i.e., system factor or repetitive member factor) is justified in designing a header. 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. 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.

TABLE 5.8 Recommended System Adjustment Factors for Header Design
Headers are not required in non-load-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 non-load-bearing walls. In the interest of added rigidity and fastening surface, however, some builders use additional jamb studs for openings in non-load-bearing walls, but such studs are not required.

**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).

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 midpoint(s); and 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.

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 pounds, depending on the steel pipe’s 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.

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 Kf accounts for the capacity reduction in bending load in nailed or bolted built-up columns. It applies, however, only to the weak-

---

### Header Type and Application

<table>
<thead>
<tr>
<th>Header Type and Application</th>
<th>Recommended C&lt;sub&gt;r&lt;/sub&gt; Value&lt;sup&gt;2&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>2x10 double header of No. 2 Spruce-Pine-Fir</td>
<td>1.30&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>Above header with double top plate, 2x10 floor band joist, and sole plate of wall located directly above&lt;sup&gt;4&lt;/sup&gt;</td>
<td>1.8</td>
</tr>
</tbody>
</table>
axis buckling or bending direction of the individual members and therefore should not be used to determine CP for column buckling in the strong-axis direction of the individual members.

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. 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.

**FIGURE 5.6 Wood Column Types**

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.

General Information

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. Rafters are repetitive framing members that support the roof sheathing and typically span from the exterior walls to a non-structural 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 article groups them under one section.

**FIGURE 5.7 Structural Elements of a Conventional Roof System**
Roof trusses are pre-engineered components. They are fabricated from 2-inch-thick dimensional 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.

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 dimensional lumber roof systems designed according to the NDS. Where appropriate, the procedure incorporates modifications of the NDS.

When inspecting roof elements or components, the inspector needs to consider the following load combinations (Table 3.1):

- \( D + (L_r \text{ or } S) \)
- \( 0.6 \, D + Wu \)
- \( D + W \)

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.

Roofs exhibit system behavior that is, in many respects, similar to floor framing; 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. Thus, a conservative system factor of 1.15 is recommended for chord bending stresses, and a factor of 1.1 for chord tension and compression stresses.
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. 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.

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, Cr, is applicable if the ridge beam is composed of two or more members. 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.

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.

Roof Trusses

Roof trusses incorporate rafters (top chords) and ceiling joists (bottom chords) into a structural frame fabricated from 2-inch-thick dimensional 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 (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.
Roof truss manufacturers normally provide the required engineering design based on the loading conditions specified by the building inspector.

The building inspector 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 inspector should also account for permanent bracing of the truss system at locations designated by the truss inspector. 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 end-wall bracing pertains to the role of the roof system in supporting the walls against lateral loads, particularly those produced by wind. Temporary bracing during construction is usually the responsibility of the contractor and is important for worker safety.

The inspector 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). 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).

**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 inspector may need to perform calculations; however, such calculations are rarely necessary in residential construction.

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. 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. More importantly, large suction pressures on roof sheathing in high wind areas 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.

**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.

A study completed 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. 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 damp, 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).

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 end-wall 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 inspector judgment should be exercised in this framing application with respect to the application of deflection criteria.

**FIGURE 5.9 Typical Roof Overhang Construction**

![Typical Roof Overhang Construction Diagram]

Table 5.6 may assist in dealing with the need to meet a reasonable serviceability limit for deflection.

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 and braces the end walls against lateral wind loads by direct attachment to rafters.

**Structural Design on Wood Framing Quiz**
The residential construction material ______ commonly used above grade in North America is light-frame wood.

- most
- least

Many elements of a home work _____ to resist lateral and axial forces imposed on the above-grade structure and transfer them to the foundation.

- together as system
- independently

A bending member is a ________ member that makes up a residential structural system.

- structural
- supplementary
- fixative

_____ is foremost a non-homogeneous, non-isotropic material, and thus exhibits different structural properties, depending on the orientation of stresses relative to the grain of the wood.

- Wood
- Steel
- Concrete

_____ are broad-leaved deciduous trees, while _____ (i.e., conifers) are trees with needle-like leaves and are generally evergreen.

- Hardwoods..... softwoods
- Softwoods..... hardwoods

Douglas fir-larch, southern yellow pine, hem-fir, and spruce-pine-fir are species groups that are widely used in ______ applications in the United States.

- residential
- commercial

Lumber is ____ 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.

- graded
- treated
- priced
- sawn
Bound water in wood is contained within the wood cells and accounts for _____ of the moisture.

- most
- little

The inspector should understand about the vertical movement that may occur in a newly-built structure as a result of ________.

- shrinkage
- enlargement
- growth
- settlement

When wood is subject to moisture levels above ____ and other favorable conditions, decay begins to set in.

- 20%
- 5%
- 10%

Typical ________ for structural wood panels specify either the maximum allowable center-to-center spacing of supports, 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).

- span ratings
- fly ratings
- corrective ratings
- measurement ratings

For bending members bearing on wood or metal, a minimum bearing of ____ is typically recommended.

- 1.5 inches
- 0.5 inches
- 3 inches
- 12 inches

For bending members bearing on masonry, a minimum bearing of ____ is typically advised.

- 3 inches
- 1.5 inches
- 0.5 inches
- 12 inches

Relatively few members in light-frame construction resist ______ forces only.
For floor joist spans less than 15 feet, a deflection limit of ____ considering design live loads only may be used, where is the clear span of the joist in inches.

- /360
- /480
- /500
- /180

For floor joist clear spans greater than 15 feet, the maximum deflection should be limited to ____.

- 0.5 inches
- 0.25 inches
- 0.1 inches
- 552 mm

T/F: As an additional recommendation, glue and mechanical fastening of the floor sheathing to the floor joists can enhance the floor system’s strength and stiffness.

- True
- False

T/F: A strong-back is a continuous bracing member, typically a 2x6, fastened edgewise to the base of the vertical web of each truss with two 16d nails.

- True
- False

Structural Design of Roof Framing

Roof Styles

In this section, we'll be using common terms — the same things can have different names in different parts of North America. The term “roof-covering materials” refers only to the visible roof-covering material, such as the shingles, tile, metal or slate, which form the primary roof covering. It doesn't include other roofing materials such as underlayment or flashing. The term “roofing materials” includes everything attached to the roof deck.

We're going to start by identifying some basic roof styles and features.

Gable
Gable roofs are one of the most common styles. They’re easily identified. They have two planes and the ridge extends the length of the home. The lower, level edges of the roof are called the “eaves,” and the sloped edges are called the “gables” or “rakes.” (We use both terms.)

**Hip**

There are two types of hip roofs, and both have four planes. The basic hip roof has a level ridge, but the ridge doesn’t extend all the way to the exterior walls. Instead, hip rafters slope diagonally down to each corner.

The photo above shows a “full hip” roof. Full hip roofs have no real ridge. The hip rafters all meet to form a point at the peak of the roof.

**Mansard**

Mansard roofs were invented by the French when owners were taxed by the height of the building as measured to the roof eave. They’re short, steep roofs installed around the perimeter of what’s usually (but not always) a flat-roofed building.
Some of these roofs are nearly vertical, and this can cause installation problems which will vary with the different types of roof-covering materials.

**Flat**

Flat roofs have one plane and very little slope. A typical slope would be ¼-inch per foot.

Flat roofs may drain over the roof edges or through scuppers installed in a parapet wall built around the perimeter.

Flat roofs are low-slope roofs. Since this series focuses on steep-slope roofs, we won’t be talking much about flat roofs. Low-slope and steep-slope roofs have different requirements.

**Shed**
Shed roofs have one plane but more slope than a flat roof. Because shed roofs are often used for additions, one potential problem area is along the upper edge of the shed roof where it ties into the wall of the original home.

Gambrel

Gambrel roofs are usually associated with barns but are not uncommon on homes. They have two planes, each of which changes slope in a convex manner. The point at which the roof changes slope should have metal flashing.

This barn is located in an area so windy that whenever the wind stops blowing, all the chickens fall over.

Bonnet

Bonnet roofs have a change of slope but are concave — the opposite of a gambrel.
Butterfly Roof

This is a style seen less often, but you will see them occasionally. When you inspect a home with a butterfly roof, look closely at the ceiling and floor beneath the low point.

The house in this photograph had recently sold and the sellers had hired a contractor to install a new roof. The buyers moved in... it rained... and the roof leaked. The buyers had to hire both a (different) roofing contractor and a floor contractor.

The roof wasn’t likely to leak due to the design alone, so this well-known architect designed not one, but two penetrations into the low point. The only things lacking are an anchor and a bilge pump!

ROOF FEATURES

Clerestory

These photos show roofs with clerestory windows. Although the term “clerestory” refers to the position of the windows, it also generally describes their position as incorporated into a shed roof. In other words, “clerestory” is commonly used to refer to the combination of roof and windows.
Clerestory windows should have adequate clearance between the sills and the roof below in areas with heavy snowfall. This home doesn’t and is more likely to leak. They should also have proper sidewall flashing.

**Cupola**

Cupolas are small structures built into the peak of a roof, often to provide light to the area below. The inspection concern is the roof framing supporting the cupola. Although the framing will typically be hidden behind interior wall-covering materials, look for signs of movement, such as cracking. Other vulnerable areas are headwall and sidewall flashing.

**Conical Roofs**

Conical roofs are often used to cover towers, as you see here, and are often steep. This first photograph shows a conical roof that is actually a series of tapered flat roofs, creating a series of hips.
In this photograph, you can see that four tiny dormers have been installed near the peak.

Inspecting these steep roofs closely is difficult (or impossible) without special equipment, so you should get as close as you can using binoculars to look for signs of leakage beneath these roofs.

Inspection concerns include flashing at the round sidewalls and areas at which conical roofs intersect with roofs of other shapes. Specially shaped crickets or flashing may be needed to provide long-term protection against leakage. Crickets are shown here outlined in red.

These areas of intersection (which are difficult to see because they’re on the backside of the roof) often collect debris, such as leaves and sediment. This debris holds moisture against the roof and flashing, which often corrodes more quickly than on the rest of the roof. So, the areas of intersection and their weak points are difficult to see.

If you can’t confirm the condition of the roofing on the backside of a conical roof, you need to disclaim it and recommend inspection by a qualified roofing contractor. A contractor may need to hook a ladder over the ridge in order to get high enough on the roof to see the backside of a conical roof clearly. This is especially true when the roof is covered with fragile materials, such as slate or tile.
Dormers

Dormers are projections built into the plane of a roof. Here, you see dormers with gable, hip and shed roofs. Inspection concerns are valleys, headwall and sidewall flashing.

Other Roof Combinations and Styles

You’ll often see several roof styles combined on one home… and sometimes…

...you’ll see roof styles for which there really is no name.

The structure above is a dormer because it’s a projection built into the plane of a roof. The structure below is a second story, since the exterior wall is continuous from foundation to roof.

The only limitations to the number of styles possible are the human imagination, the laws of physics, and the depth of the homeowner’s pockets.

Each different style of roof and roof feature has its weak points. Once you learn what these are, you’ll know where to expect problems. With all roofs, weak points are:

- places where roof-covering materials change;
• places where the roof changes direction;
• places where materials are used that have a relatively short lifespan;
• roof penetrations; and
• portions of the roof that lie in the drainage path.

Roof Framing, Part 1

You’ll be evaluating the roof framing from inside the attic space, but we have an advantage in technology. Let’s strip away the roof and wall coverings of a home and identify some of the more common roof framing members. We’ll start with a conventionally framed roof in which individual roof-framing members are cut and assembled on-site.

Conventional Roofs

Common Rafters

Rafters which rest on the outside walls at the bottom and connect to the ridge at the top are called “common rafters” (highlighted here in yellow).

Rafters on opposite sides of the ridge should be installed directly opposite each other in pairs — although, if you see a few that don’t align, it’s really not a defect. Rafters sometimes have to be moved a little to accommodate components of other home systems. The illustration above shows a rafter moved to accommodate a combustion vent.

If you see many rafters that don’t align, you may comment on this, but in existing homes, refrain from calling it a defect unless you see failure. In newer homes, many rafters which don’t oppose usually indicate poor-quality framing. It’s an indication that you should look carefully for other problems in the roof framing.
Rafters are typically installed on 24-inch centers. If you see rafters installed on centers greater than 24 inches, look for signs of failure, such as sagging of the rafters. If you see sagging rafters, recommend stabilization by a qualified contractor. Stabilization typically involves installation of a purlin system.

**Hips**

Hip roofs have “hip rafters” which are oriented diagonally to the ridge and outside walls. Hip rafters are simply called “hips,” and are shown here as brown. Hips rest on an outside corner at the bottom and connect to the ridge at the peak.

Rafters which rest on the exterior walls at the bottom and connect to a hip at the top are called “hip jacks,” shown here as purple.

**Valleys**

Where ridges change direction, an inside corner is created, which is spanned by a “valley rafter” or simply “valley,” shown here as green. Valleys are also oriented diagonally to the ridge and exterior walls. Valleys rest on top of the walls at the inside corner at the bottom, and connect to the ridge at the top.

Rafters which connect to the valley at their bottoms and connect to the ridge at the top are called “valley jacks,” shown here as light blue.

**Conventional Ridge**

The illustration shows a conventional ridge (colored orange). In homes with conventional ridges, the rafters support the weight of the roof and transmit the roof load down through
the walls to the foundation and, finally, to the soil. The route taken by the weight of the roof through the framing members to the soil is called the “load path.”

The purpose of the ridge is to provide an easy method for connecting rafters at the peak of the roof, and to provide better nailing at the peak.

Older homes may have no ridge at all. That was a common building practice at one point in various parts of North America, and it’s not a defect as long as the rafters oppose each other.

Engineered lumber used for roof framing has very specific requirements for connections, and discussing them here exceeds the scope of this series. The manufacturers of metal connectors for engineered lumber publish connection specifications in their catalogues and on their websites.

**Rafter Ties**
In homes with flat ceilings and an attic space, the bottoms of opposing rafters should be fastened together with ceiling joists, which form “rafter ties.” When rafters have been installed perpendicular to the ceiling joists, rafter ties typically rest on top of the ceiling joists.

Rafter ties prevent the weight of the roof from spreading the tops of the walls and causing the ridge to sag.

**Collar Ties**

Collar ties connect the upper ends of opposing rafters. They should be installed on every other rafter in the upper third of the roof. Their purpose is to prevent uplift. Whether or not they should be installed is an engineering call. They aren’t always required so the lack of them is not a defect, but when you see them, they should be installed correctly.

Here, you can see collar ties installed in the upper third of the roof, and rafter ties installed down low and spliced over a wall.

**Purlin Systems**
You can also see the purlin system. Purlin systems are designed to reduce the distance that rafters have to span. They consist of strongbacks nailed to the undersides of the rafters and supported by diagonal braces.

The bottoms of purlin braces should rest on top of a bearing wall. Braces that rest on ceiling joists or which somehow pass the roof load to the ceiling below are defective installations. If you see braces which rest on ceiling joists, look for a sag in the ceiling.

Braces are typically installed every other rafter and should be at an angle no steeper than 45°.

Here’s a purlin system installed in the garage of an older home. With no central wall to carry the braces, they bear on a strongback that rests on the ceiling joists. There was no sagging, so there was no comment in the inspection report.

Purlin systems have been built in many ways — some better than others. Modern building codes call for strongbacks to be of equal or greater dimension than the rafter dimension, but most purlin strongbacks you’ll see will not meet this requirement. If you know that the home was required to meet this code when it was built, call it a defect; otherwise, limit your inspection to looking for signs of failure, such as sagging or broken rafters and broken components. Also, look for improper installations, such as braces resting on ceiling joists, braces but no strongback, and too few braces.

In older homes in some areas, it’s common to find no strongbacks. It’s a quality issue unless the roof is sagging; then, it’s a structural issue and you should recommend stabilization by a qualified contractor.
The term “purlin” has several different meanings depending on what part of North America you’re in, what part of the roof you’re talking about, and the background of the person you’re discussing it with, so don’t be surprised if someone tries to correct you.

**Structural Ridge**

Homes with vaulted ceilings usually don’t have rafter ties to keep the walls from spreading and the ridge from sagging, so they use a structural ridge. In a home with a structural ridge, the ridge consists of a beam strong enough to support the roof load without sagging.

**Overframe**

When you’re inside an attic, you may see a condition in which the ridge and a few jack rafters from one roof section are framed on top of an existing roof.

This is called an “overframe” and it’s quite common in certain areas. Built correctly, it’s structurally sound.
You’ll often see a section of roof sheathing removed to provide a passageway between attic spaces. If you can’t enter a portion of the attic, recommend that it be inspected by a qualified inspector after access is provided. This is especially important if it contains plumbing or electrical components.

**Roof Framing, Part 2**

**Metal Connections and Fasteners**

Rafters may be connected with metal hardware or just nailed to the ridge. Rafters on one side of the ridge will be nailed through the ridge, and those nails will be hidden behind the opposing rafters. The opposing rafters will be toe-nailed. The proper nailing schedule for toe-nailing rafters is three nails in one side and two in the other.

In roof framing, there are a lot of places where framing members connect. Requirements for these connections have changed over the years, but you can still identify basic defects.

Structural engineers have to calculate the loads on connections between framing members and specify hardware that will support those loads. Fasteners are what attach metal connectors to wood framing members. In order to ensure safe connections, fasteners of the right metal alloy and of the correct minimum diameter and length have to be used.

When a workman uses a roofing nail instead of a hanger nail at a structural connection, that connection will be much weaker because roofing nails are weaker than hanger nails. Roofing nails are designed to anchor roofing materials against uplift, not to support a structural load. If fasteners are used that are inadequate in strength, the connection may fail.

**Fastener Failure**

Fasteners can fail in one of two ways: withdrawal or shear.
Withdrawal simply means that the fastener pulls out. When withdrawal causes failure, the force causing the failure is parallel to the shaft of the fastener.

Shear failure is caused by a force perpendicular to the shaft of the fastener. The fastener bends and breaks as if it had been sheared off by a guillotine.

It's important that you be able to identify proper fasteners. The following are all acceptable, but the most commonly used, acceptable nails are 16-penny (16d) checker-head, or #8d and #10d hanger nails. Of the two 10d shown in the photos below, the first one is galvanized, and the second one is not galvanized.
Although any nail with a number cast into the head is acceptable, not all acceptable nails are numbered, so look closely. Acceptable nails tend to have thicker heads.

These are examples of nails NOT ACCEPTABLE for use with metal connectors. Finding the 8-penny, checker-head sinker installed is an especially common defect.

Building department officials often pass structures in which many connectors were fastened with 8-penny, vinyl-coated sinkers, but they shouldn’t have. Eight-penny, vinyl-coated sinkers used with metal connectors are a defective installation. So many connectors have been installed with 8-penny, vinyl-coated sinkers without being called out as a violation by building department officials that you should not recommend replacement unless you find them on heavy steel connectors. Instead, recommend evaluation by a structural engineer and let him be the one to jam the crowbar into the spokes of the transaction. He may also say it’s fine, but you should pass the liability on to the engineer.
Roof trusses are engineered roof framing systems in which the main components — roof trusses — are designed by structural engineers, then assembled in a manufacturing facility before being delivered to the job site by truck.

Let's take a look at how trusses are built.

Trusses are manufactured in a wide variety of configurations and have been around since the early 1950s. Trusses have to be engineered correctly, so if you see trusses fastened together with plywood gussets instead of rings or gangnails...
... you're looking at a non-professional design, and you should recommend evaluation by a structural engineer.

In this photo of the same home, you can see that roof leakage has caused wood decay of the plywood gusset. By the time decay becomes visible, wood may have lost up to 50% of its strength, so decay is one more reason to recommend evaluation by a structural engineer.

Most roof trusses are designed to bear on the exterior walls only. Trusses touching interior walls can transfer roof loads to walls not designed to carry a structural load.

Trusses touching interior walls can also create point loads on trusses at points not designed to support point loads. In rare cases, this has resulted in “exploding trusses.”

As you can see in the image above, the bottom chords of trusses should be fastened to the tops of interior, non-bearing walls with slotted clips which allow for some vertical movement of the trusses. Movement is usually related to changes in the moisture content of the wood trusses. This can be a response to changes in relative humidity or other conditions which cause moisture level fluctuations in attic spaces.

Truss movement can also result when roof loads exceed the structural design loads of the trusses, as might happen with the accumulation of lots of wet, heavy snow in an area that seldom gets snow.
Trusses are usually braced with a system of 2x4s and 1x6s when they're installed. The locations of bracing can be different for different truss designs, and you'll have no way of knowing what the requirements are. Trusses are often installed with blocks at the roof peak and above the outside walls, but these are not always required. So, in your report, don’t call missing blocks or bracing a defective condition.

Look for signs of failure. Trusses out of plumb are poor-quality construction but may be stable. If they’re badly out of plumb, mention that in your inspection report. Look for broken or damaged truss components, and comment on them in your report.

Trusses should never, ever be structurally altered in any way without approval from a structural engineer. If you see trusses which have been cut or reinforced, recommend evaluation by a structural engineer.

Trusses sometimes rest in hangers instead of bearing on a wall. When this is the case, check the fasteners carefully. These hangers were fastened with roofing nails, and that’s a defective installation.
Here's the garage of the house. The neighbor told the inspector that the roof of the garage next door had collapsed during a big snowstorm the previous year.

It's easy to see that the trusses have been altered. Plywood gussets were added at a connection that would typically have had metal gangnails installed.

In the rare instances in which alterations involving plywood gussets have been approved by a structural engineer, gussets usually have backing for perimeter nailing installed, are glued with a special construction adhesive (such as PL Premium), and are heavily nailed, with nails every two or three inches or so. You should see lots of nails and glue squeezing out of joints. As you can see in the photo above, that wasn't the case here.

Looking over to the wall, notice that the hangers seem to be small for the load they're carrying.
The hangers turned out to be sized for a 2x4, which is far too small for the roof load they are carrying. They were fastened with a total of four gold deck screws each! The deck screws are a serious defect, rated far below acceptable hanger nail strength.

In addition to that, they were installed through drywall, which does not support the shaft of a fastener the way wood does.

The problems don't end there. If you look closely at the gangnail, you can see that it has been damaged and the spikes are no longer embedded in the wood. Instead, the gangnail is attached by a couple of nails which have been bent over.

This roof is structurally inadequate and dangerous. It needs to have corrections designed by a structural engineer, and bids from qualified contractors for making the corrections. Corrections needed to be completed as soon as possible.

**Structural Design of Roof Framing Quiz**

T/F: The term “roof-covering materials” refers only to the visible roof-covering material, such as the shingles, tile, metal or slate, which form the primary roof covering.

- True
- False

____ roofs are one of the most common styles, easily identified, and they have two planes and the ridge extends the length of the home.

- Gable
- Hip
- Mansard
- Flat

A full ____ roof has four planes, no ridge board, and a peak.

- hip
- gable
- flat
- mansard
Rafters which rest on the outside walls at the bottom and connect to the ridge at the top are called “____ rafters.”

- common
- hip
- jack

**Structural Design of Lateral Resistance to Wind and Earthquake**

**General Information**

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.

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 article focuses on methods for evaluating the lateral resistance of individual sub-assemblies 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 toolbox. 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 article provide a useful guide and resource that
supplement existing building code provisions. More importantly, the article 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 inspector 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. 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.

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.
Therefore, the inspector should have a reasonable knowledge of the underpinnings of current LFRS design approaches (including their uncertainties and limitations). However, many inspectors 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. To this end, the next section presents a brief overview of several whole-house lateral load tests.

**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.

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 “post-yielding” 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.

**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.

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.
3. Distribute shear loads to the LFRS (wall, floor, and roof systems).
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.
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, 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.

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 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.

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 2, 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 1).

Figure 1. Chords in Shear Walls and Horizontal Diaphragms Using the Deep Beam Analogy
Hold-down restraints are devices used to restrain the whole building and individual shear wall segments from the overturning that results from the levering (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 3). In many typical residential
applications, however, overturning forces may be resisted by the dead load and the contribution of many component connections (see Figure 3). Unfortunately (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 misapplications.

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.
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.

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 4 for a continuous horizontal diaphragm (i.e., floor) with three supports (i.e., shear walls).

Figure 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. In wind design, the tributary areas are associated with the lateral component of the wind load acting on the exterior surfaces of the building.

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, the actual stiffness of these systems is extremely difficult to predict accurately. 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. If this assumption were more accurate, the interior shear wall would be over-designed and the exterior shear walls under-designed 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**

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 under-design (i.e., weak or "soft" story) can wreak havoc on a building and constitute a serious threat to life.

**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 1 and 3). 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 nonlinearity 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 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 3). 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) 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. Figure 5 illustrates a perforated shear wall.

**Figure 5. Illustration of a Basic Perforated Shear Wall**

![Figure 5. Illustration of a Basic Perforated Shear Wall](image)

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 being equal —
can yield a stronger wall. Conversely, a PSW design with increased sheathing fastening
can out-perform 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 (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).
Basic Diaphragm Design Approach

As described earlier in this article, 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 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.

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).

Design Guidelines

General Approach

This section outlines methods for designing shear walls and diaphragms. 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 are as follows:

- \(0.6D + (W \text{ or } 0.7E)\) ASD
- \(0.9D + (1.5W \text{ or } 1.0E)\) 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. 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 (say, 10 percent). Indeed, the seismic shear load using the simplified method 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.

Shear Wall Design

Shear Wall Design Values (Fs)

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 article are appropriately adjusted to ensure an acceptable safety margin.

Wood Structural Panels (WSP)

Table 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 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 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 1. Unfactored (Ultimate) Shear Resistance (plf) for Wood Structural Panel Shear Walls with Framing of Douglas Fir, Larch, or Southern Pine**

<table>
<thead>
<tr>
<th>Panel Grade</th>
<th>Nominal Panel Thickness (inches)</th>
<th>Minimum Nail Penetration in Framing (inches) (APA, 1998)</th>
<th>Nail Spacing at Panel Edges (inches)</th>
<th>Nail Size (common or galvanized)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Structural</td>
<td>5/8</td>
<td>1-1/4</td>
<td>6d</td>
<td>621</td>
</tr>
<tr>
<td>Larch</td>
<td>3/4&quot;</td>
<td>1-1/8</td>
<td>6d</td>
<td>633</td>
</tr>
<tr>
<td>Southern</td>
<td>7/8&quot;</td>
<td>1-1/8</td>
<td>6d</td>
<td>665</td>
</tr>
<tr>
<td>Pine</td>
<td>11/32&quot;</td>
<td>1-1/8</td>
<td>6d</td>
<td>757</td>
</tr>
<tr>
<td></td>
<td>15/32&quot;</td>
<td>1-1/2</td>
<td>10d 🅱️</td>
<td>1,256</td>
</tr>
</tbody>
</table>

The unit shear values in Table 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 Lightweight 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 1, including the recommended safety factor or resistance factor. Table 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 2. Unfactored (Ultimate) Unit Shear Resistance (plf) for Walls with Cold-Formed Steel Framing and Wood Structural Panels**

<table>
<thead>
<tr>
<th>Panel Grade</th>
<th>Panel Type and Nominal Thickness (inches)</th>
<th>Minimum Screw Size</th>
<th>Screw Spacing at Panel Edges (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Structural 1</td>
<td>7/16 OSB</td>
<td>#8</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td>15/32 plywood</td>
<td>#8</td>
<td>780</td>
</tr>
</tbody>
</table>

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). 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 3 provides estimated ultimate unit shear values for gypsum wall board sheathing.

**TABLE 3. Unfactored (Ultimate) Unit Shear Values (plf) for 1/2-Inch-Thick Gypsum Wall Board Sheathing**

<table>
<thead>
<tr>
<th>GWB Thickness</th>
<th>Blocking Condition</th>
<th>Spacing of Framing (inches)</th>
<th>12</th>
<th>8</th>
<th>7</th>
<th>6</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2 inch</td>
<td>Blocked</td>
<td>16</td>
<td>120</td>
<td>210</td>
<td>250</td>
<td>260</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>Unblocked</td>
<td>16</td>
<td>80</td>
<td>170</td>
<td>200</td>
<td>220</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24</td>
<td>40</td>
<td>120</td>
<td>150</td>
<td>180</td>
<td>220</td>
</tr>
</tbody>
</table>

**1x4 Wood Let-In Braces and Metal T-Braces**

Table 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 4. Unfactored (Ultimate) Shear Resistance (lbs) for 1x4 Wood Let-Ins and Metal T-Braces**

<table>
<thead>
<tr>
<th>Type of Diagonal Brace</th>
<th>Ultimate Horizontal Shear Capacity (per brace)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1x4 wood let-in brace (8-foot wall height)</td>
<td>600 lbs (tension and compression)</td>
</tr>
<tr>
<td>Metal T-brace</td>
<td>1,400 lbs (tension only)</td>
</tr>
</tbody>
</table>
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 U.S. 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 particleboard and fiberboard. Ultimate unit shear values for fiberboard 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 fiberboard and other materials’ shear resistance qualities. In one study that conducted tests on various wall assemblies for HUD, fiberboard 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 non-additive. 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 fiberboard (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, 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.

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.
Perforated Shear Wall Design Approach

The following equations provide the design shear capacity of a perforated shear wall:

\[ F_s = (F_s' K_k C_w (L/3SF) (\sigma_p)) \]  
\[ F_{ps} = (F_s') K_k C_w (L) \]

where:
- \( F_s' \) = the factored and adjusted design unit shear capacity (plf) for the wall construction
- \( F_s \) = the unfactored (ultimate) and unadjusted unit shear capacity (plf)
- \( K_k \) = an adjustment factor applied on the wood structural panel sheathing \( F_s' \) values in accordance with Section 6.5.2.3
- \( C_w \) = 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
- \( 3/SF \) = the safety factor adjustment for use with ASD
- \( \sigma_p \) = the resistance factor adjustment for use with LRFD

The PSW method has the following limits on its use:

- The value of \( F_s \) for the wall construction should not exceed 1,500. The wall must be fully sheathed with wood structural panels on at least one side. Unit shear values of sheathing materials may be combined.

- 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. 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.

- Top plates must be continuous with a minimum connection capacity at splices with lap joints of 1,000 lbs., 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 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 \( F_{ScspCns} \) 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). For wall constructions with ultimate shear capacities \( F_{ScspCns} \) 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:

\[
F_s = F_{c0} C_0 C_1 \frac{1}{SF} \quad \text{or} \quad \phi
\]

\[
F_{m} = F_s \times [L, 1]
\]

where,

- \(F_{c0}\) = 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 \(C_0\) and \(C_1\) adjustment factors apply only to wood structural panel sheathing \(F_s\) values
- \(F_{m}\) = 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
- \(L\) = the length of a shear wall segment (total width of the sheathing panel(s) in the segment)
- \(SF\) = the safety factor adjustment for use with ASD
- \(\phi\) = the resistance factor adjustment for use with LRFD

The segmented shear wall design method 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. 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.
- 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 load-deformation equations.

Shear Capacity Adjustment Factors

Safety and Resistance Factors (\(SF\) and \(\phi\))

Table 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 5. Minimum Recommended Safety and Resistance Factors for Residential Shear Wall Design**

<table>
<thead>
<tr>
<th>Type of Construction</th>
<th>Safety Factor (ASD)</th>
<th>Resistance Factor (LRFD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detached garages and ancillary buildings not for human habitation</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Single-family houses, townhouses, and multifamily low-rise buildings (apartments)</td>
<td>Seismic 2.5, Wind 2.0</td>
<td>Seismic 0.55, Wind 0.7</td>
</tr>
</tbody>
</table>

**Species Adjustment Factor (Csp)**

The ultimate unit shear values for wood structural panels in Table 1 apply to lumber species with a specific gravity (density) $G$ greater than or equal to 0.5. Table 6 presents specific gravity values for common species of lumber used for wall framing. For $G$ less than 0.5, the following value of Csp should be used to adjust values in Table 1 only (APA, 1998):

$$Csp = [1 - (0.5 - G)] \times 1.0$$

**TABLE 6. Specific Gravity Values (Average) for Common Species of Framing Lumber**

<table>
<thead>
<tr>
<th>Lumber Species</th>
<th>Specific Gravity, $G$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern Yellow Pine (SYP)</td>
<td>0.55</td>
</tr>
<tr>
<td>Douglas Fir-Larch (DF-L)</td>
<td>0.60</td>
</tr>
<tr>
<td>Hem-Fir (HF)</td>
<td>0.43</td>
</tr>
<tr>
<td>Spruce-Pine-Fir (SPF)</td>
<td>0.42</td>
</tr>
</tbody>
</table>

**Nail Size Adjustment Factor (Cns)**

The ultimate unit shear capacities in Table 1 are based on the use of common nails. For other nail types and corresponding nominal sizes, the Cns adjustment factors in Table 7 should be used to adjust the values in Table 1. Nails should penetrate framing members a minimum of 10D, where D is the diameter of the nail.

**TABLE 7. Values of Cns for Various Nail Sizes and Types**

<table>
<thead>
<tr>
<th>Nominal Nail Size (penny weight)</th>
<th>Nail Length (inches)</th>
<th>Nail Type</th>
<th>Pneumatic (by diameter in inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d</td>
<td>1-7/8 to 2</td>
<td>Common¹</td>
<td>0.092, 0.113, 0.131, 0.148</td>
</tr>
<tr>
<td>8d</td>
<td>2-3/8 to 2-1/2</td>
<td>Box¹</td>
<td>0.9, 0.8, 0.5, 0.75, 1.0, N/A²</td>
</tr>
<tr>
<td>10d</td>
<td>3</td>
<td>Pneumatic</td>
<td>N/A², N/A², 0.8, 1.0</td>
</tr>
</tbody>
</table>

**Opening Adjustment Factor (Cop)**
The following equation for Cop applies only to the perforated shear wall method:

\[ \text{Cop} = \frac{r}{(3-2r)} \]

where

\[ r = \frac{1}{1 + \frac{\alpha}{\beta}} = \text{sheathing area ratio (dimensionless)} \]

\[ \alpha = \frac{\Sigma A_o}{(H \times L)} = \text{ratio of area of all openings } \Sigma A_o \text{ to total wall area, } H \times L \text{ (dimensionless)} \]

\[ \beta = \frac{\Sigma L_i}{L} = \text{ratio of length of wall with full-height sheathing } \Sigma L_i \text{ to the total wall length } L \text{ of the perforated shear wall (dimensionless)} \]

**Dead Load Adjustment Factor (Cdl)**

The Cdl factor applies to the perforated shear wall method only. 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. The Cdl adjustment factor is determined as follows and should not exceed 1.15:

\[ C_{dL} = 1 + 0.15 \frac{w_D}{300} \times 1.15 \]

where

\[ w_D = \text{the net uniform dead load supported at the top of the perforated shear wall (plf) with consideration of wind uplift and factoring.} \]

**Aspect Ratio Adjustment Factor (Car)**

The following Car adjustment factor applies only to the segmented shear wall design method for adjusting the shear resistance of interior and exterior sheathing:

\[ C_a = \frac{1}{\sqrt{0.5a}} \text{ for } 2.0 \leq a \leq 4.0 \]

\[ C_a = 1.0 \text{ for } a < 2.0 \]

where

\[ a = \text{the aspect ratio (height/width) of the sheathed shear wall segment.} \]

**Overturning Restraint**

214
Figure 3 addressed 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;
- 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, 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, non-engineered residential construction. Mechanics-based methods to assist in the more complicated design approach are under development.

Using basic mechanics as shown in Figure 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:

\[ M_c = 0 \]
\[ F_r (d/2) - T (x) - D_w (d/2) - (w_o)(d/2) = 0 \]

\[ T = \frac{d}{x} \left( F_r h + \frac{1}{4} D_w - \frac{1}{2} (w_o)(d) \right) \]

\[ M_T = 0 \]

\[ C = \frac{d}{x} \left( F_r h + \frac{1}{4} D_w + \frac{1}{4} (w_o)(d) \right) \]

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

h = the height of the wall (ft)

Dw = the dead load of the shear wall segment (lb); dead load must be factored and wind uplift considered.

wD = the uniform dead load supported by the shear wall segment (plf); dead load must be factored and wind uplift considered.

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:

\[ T = C = \frac{d}{x} F's \cdot h \]

Any tension or compression force transferred from shear wall overturning forces
originating above the wall under consideration must be added to the result 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.

**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 \((8 \text{ ft})(F'_{s}\))/(12 \text{ 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).

**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. The degree of over-prediction 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 article. It provides a nonlinear load-drift relationship up to the ultimate capacity of the perforated shear wall. 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).

\[
\Delta = 1.8 \frac{0.5}{G} \frac{1}{\sqrt{R}} \frac{V_d}{F_{P_{SW,ULT}}} \frac{2.8}{8} \frac{h}{8} \text{ (inches)}
\]

where

\( \Delta \) = the shear wall drift (in) at shear load demand, \( V_d \) (lb)

\( G \) = the specific gravity of framing lumber (see Table 6)

\( R \) = the sheathing area ratio

\( V_d \) = 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 \( F_{P_{SW,ULT}} \) while the value of \( V_d \) should be set to the design shear load when checking drift at design load conditions

\( F_{P_{SW,ULT}} \) = the unfactored (ultimate) shear capacity (lb) for the perforated shear wall (i.e., \( F_{P_{SW}} \times SF \) or \( F_{P_{SW}}/\phi \) for ASD and LRFD, respectively)

\( h \) = the height of wall (ft)

**Segmented Shear Wall Load-Drift Relationship**
APA Semi-Empirical 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:

$$
\Delta = 2.2 \frac{0.5}{G} \sqrt[4]{a} \frac{V_d}{F_{SSW,ULT}} \left( \frac{h}{8} \right)^{2.8} \text{ (in)}
$$

where

$\Delta =$ 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., $F_{SSW} \times SF$ or $F_{SSW}/\phi$ 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.

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 pre-engineered portal frame and shear wall elements that can be ordered to custom requirements or standard conditions.

**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 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. 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 8. Horizontal Diaphragm ASD Shear Values (plf) for Unblocked Roof and Floor Construction Using Douglas Fir or Southern Pine Framing**

<table>
<thead>
<tr>
<th>Panel Type and Application</th>
<th>Nominal Panel Thickness (inches)</th>
<th>Common Nail Size</th>
<th>Design Shear Values (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural 1 (Roof)</td>
<td>5/16</td>
<td>6d</td>
<td>165</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>6d</td>
<td>185</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td>10d</td>
<td>285</td>
</tr>
<tr>
<td>APA Stud-I-Floor (Floor)</td>
<td>7/16</td>
<td>8d</td>
<td>230</td>
</tr>
<tr>
<td>and Rated Sheathing</td>
<td>15/32</td>
<td>8d</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>19/32</td>
<td>10d</td>
<td>285</td>
</tr>
</tbody>
</table>
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:

\[
V_{\text{max}} = \frac{1}{2} wl
\]

\[
v_{\text{max}} = \frac{V_{\text{max}}}{d}
\]

where

\(V_{\text{max}}\) = 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)

\(v_{\text{max}}\) = 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:

\[
M_{\text{max}} = \frac{1}{4} w l^2
\]

\[
T_{\text{max}} = C_{\text{max}} = \frac{M_{\text{max}}}{d}
\]

where

\(M_{\text{max}}\) = 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)

\(T_{\text{max}}\) = the maximum chord tension force (lb)

\(C_{\text{max}}\) = the maximum chord compression force (lb)
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.

**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 floor 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.

**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

\[ d = \text{the depth or width of the diaphragm in the direction of the load (ft)} \]
shear wall drift calculations. The stiffness of floor and roof diaphragms is highly dependent on the final construction, including interior finishes.

**Structural Design of Lateral Resistance Quiz**

T/F: Lateral resistance to wind and earthquake involves shear walls, diaphragms, and interconnections.

- True
- False

T/F: Lateral resistance to wind and earthquake will not help prevent building collapse.

- False
- True

T/F: 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.

- True
- False

There ____ a single design methodology or theory that provides reasonable predictions of complex, large-scale system behavior in conventionally built or engineered light-frame buildings.

- is not
- is

The nonstructural components in conventional housing (i.e., sidings, interior finishes, interior partition walls, and even windows and trim) can account for ____ of a building’s lateral resistance.

- more than 50 percent
- more than 80 percent
- less than 25 percent
- exactly 13 percent

____ are the members (or a system of members) that form a ____ to resist the tension and compression forces generated by the beam action of a diaphragm or shear wall.

- Chords? flange
If adequate connection is made between the band joist and the____, then the diaphragm sheathing, band joists, and wall framing function as a composite chord in resisting the chord forces.

- wall top plate
- doorway threshold
- lag bolt
- roof ridge

The objectives in designing a building's lateral resistance to wind and earthquake forces does not include:

- defining the nature and magnitude of hazards and external forces that a building must resist to provide reasonable performance throughout the structure's useful life
- providing a system of shear walls, diaphragms, and interconnections to transfer lateral loads and overturning forces to the foundation
- preventing building collapse in extreme wind and seismic events
- providing adequate stiffness to the structure for service loads experienced in moderate wind and seismic events

In _______________, 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.

- light-frame construction
- medium-frame construction
- heavy-frame construction

Horizontal diaphragms are assemblies, such as the roof and floors, that act as deep beams by collecting and transferring ______ to the shear walls.

- lateral forces
- vertical forces
- horizontal forces
- tension and compression forces

Chords are the members (or a system of members) that form a flange to resist the ______ forces generated by the beam action of a diaphragm or shear wall.

- tension and compression
- vertical
- horizontal
- lateral
For shear walls in typical light-frame buildings, ________ forces on shear wall chords are usually considered.

- tension and compression
- vertical
- horizontal
- lateral

________ forces result from the overturning action (i.e., overturning moment) caused by the lateral shear load on the shear wall.

- tension
- compression
- horizontal
- vertical

The __________ approach is perhaps the most popular method used to distribute lateral building loads.

- tributary area approach (flexible diaphragm)
- total shear approach ("eyeball" method)
- relative stiffness design approach

The tributary area approach is reasonable when the layout of the shear walls is generally __________ with respect to even spacing and similar strength and stiffness characteristics.

- symmetrical
- asymmetrical
- aligned

The ________ approach is the second most popular and simplest of the three LFRS design methods.

- total shear approach ("eyeball" method)
- tributary area approach (flexible diaphragm)
- relative stiffness design approach

The __________ 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.

- relative stiffness design
- total shear ("eyeball" method
- tributary area (flexible diaphragm)
When 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.

- torsional
- tension and compression
- vertical
- lateral

Ultimately, which approach 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:

- relative stiffness design approach
- tributary area approach (flexible diaphragm)
- total shear approach ("eyeball" method)

The approach is well-recognized as a standard design practice and is the most widely used method of shear wall design.

- basic perforated shear wall (psw)
- segmented shear wall design
- tributary area
- basic diaphragm

The design method is gaining popularity among designers and even earning code recognition.

- basic perforated shear wall (psw) approach
- segmented shear wall
- tributary area approach
- basic diaphragm

In the case of the method, the loads can be immediately assigned to the various shear wall lines based on tributary building areas for the two orthogonal directions of loading.

- tributary area
- segmented shear wall design
- basic perforated shear wall (psw)
- tributary area
- basic diaphragm

Lateral building loads should be distributed to the shear walls on a given story by using one of the following methods except for:

- basic diaphragm
• tributary area approach
• total shear approach
• relative stiffness approach

In the case of the ______________, the assignment of loads must be based on an assumed relationship describing the relative stiffness of various shear wall lines.

• relative stiffness approach
• tributary area approach
• total shear approach
• basic diaphragm

It is common practice (and required by some building codes) for the ______ 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.

• torsional load distribution
• vertical load distribution
• lateral load distribution
• seismic load distribution

The shear wall design value is denoted by:

• Fs
• As
• Es
• Hs

The aspect ratio of wall segments should not exceed ___ (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.)

• 4
• 1
• 2
• 3

In situations with little space to include sufficient shear walls to meet required loading conditions, the designer must turn to alternatives such as:

• portal frames
• special sheathing
• wooden frames
• steel frames
Structural Connection Design

General Information

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 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 article 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.

The photo above shows an anchor bolt connecting a wooden sill plate to the top of a concrete foundation.

This article 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). Also addressed are the relevant concrete and masonry connections prescribed in accordance with the applicable provisions of Building Code Requirements for Structural Concrete (ACI-318) and Building Code Requirements for Masonry Structures.

For most connections in typical residential construction, the connection design may be based on prescriptive tables found in the applicable residential building code. Table 1 below depicts a commonly recommended nailing schedule for wood-frame homes.
The information included in this table is based on current industry practices and other sources.

The designer should 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 and/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.

### 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 fasteners noted above.

### Nails

Several characteristics distinguish one nail from another. Figure 1 depicts key features for a few types of nails that are essential for wood-frame design and construction. This section discusses some of a nail’s characteristics relative to structural design. For additional
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 typically used for toe-nailing and many other light framing connections where splitting of lumber is a concern.
- **Cooler nails** are generally similar to the nails described 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.

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 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 2. Nail Types, Sizes and Dimensions**

<table>
<thead>
<tr>
<th>Type of Nail</th>
<th>Diameter (m)</th>
<th>Length (in)</th>
<th>Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat</td>
<td>0.13</td>
<td>0.08</td>
<td>3.3</td>
</tr>
<tr>
<td>Sinker</td>
<td>0.11</td>
<td>0.08</td>
<td>3.0</td>
</tr>
<tr>
<td>Pneumatic</td>
<td>0.13</td>
<td>0.08</td>
<td>3.3</td>
</tr>
</tbody>
</table>

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 types described above; however, other head types, such as a half-round or D-shaped heads, are also common.

The shank, as illustrated in Figure 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 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 (PWFs). 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, along with 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.

**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 2 for an illustration of some typical bolt types and connections for residential use.

**FIGURE 2. Bolt and Connection Types**
In residential wood construction, bolted connections are typically limited to wood-to-concrete connections unless the home is constructed in a high-hazard wind or seismic area, and hold-down brackets are required to transfer shear wall overturning forces. 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 pre-drilled holes. If the 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 non-uniform dowel (bolt) bearing stresses and slippage of the joint when loaded. NDS 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.

**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. 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 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 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 (concrete, masonry or wood) to hold down the end of a member or assembly (i.e., shear wall).
• Strap ties are pre-punched 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 pre-punched 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 3. Specialty Connector Hardware

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.
**Wood Connection Design**

This section covers the design procedures for nails, bolts, and lag screws. The procedures are intended for allowable stress design (ASD) such that loads should be determined accordingly. Other types of fastenings are addressed by the National Design Specification for Wood Construction (NDS) but are rarely used in residential wood construction. The applicable sections of the NDS related to connection design as covered in this course 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, nonlinear 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 3 provides the specific gravity values of various wood species typically used in house construction.

**TABLE 3. Common Framing Lumber Species and Specific Gravity Values**

<table>
<thead>
<tr>
<th>Lumber Species</th>
<th>Specific Gravity, G</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern Pine (SP)</td>
<td>0.55</td>
</tr>
<tr>
<td>Douglas Fir-Larch (DF-L)</td>
<td>0.50</td>
</tr>
<tr>
<td>Hem-Fir (HF)</td>
<td>0.43</td>
</tr>
<tr>
<td>Spruce-Pine-Fir (SPF)</td>
<td>0.42</td>
</tr>
<tr>
<td>Spruce-Pine-Fir (South)</td>
<td>0.36</td>
</tr>
</tbody>
</table>

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 damp lumber can decrease by as much as 50% over time as the lumber dries to its equilibrium moisture content (EMC). 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 a 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 these factors, there are a number of uncertainties in the design of connections that can lead to conservative or less conservative 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 those allowed by the NDS.

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 direction of the wood’s grain in each of the connected members. Refer to Figure 4 for an illustration of various connection types and loading conditions.

**FIGURE 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⊥** – shear perpendicular to wood grain; and
- **Z∥** – shear parallel to wood grain.

In addition to the already tabulated design values for the structural resistance properties of connections described above, 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.

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 conservative.

Adjusted Allowable Design Values

Design values for wood connections are subject to adjustments in a manner similar to that required for wood members themselves. 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).

\[
Z = ZC_dC_wC_kC_pC_a \quad \text{for bolts}
\]
\[
Z = ZC_dC_wC_kC_pC_{te} \quad \text{for lag screws}
\]
\[
Z = ZC_dC_wC_kC_hC_{te}C_{bi} \quad \text{for nails and spikes}
\]
\[
W = WC_dC_mC_pC_{bi} \quad \text{for nails and spikes}
\]
\[
W = WC_dC_mC_{te} \quad \text{for lag screws}
\]

The adjustment factors and their applicability to wood connection design are briefly described as follows:

- CD – Load Duration Factor (NDS•2.3.2) – 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).
- CM – 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.
- Ct – Temperature Factor (NDS•7.3.4) – applies to the W and Z values for all connections exposed to sustained temperatures of greater than 100° F; not typically used in residential framing.
• Cg – 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).
• CΔ – 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.
• Cd – 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.
• Ceg – 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 (Ceg=0.67).
• Cdi – 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 (Cdi = 1.1).
• Ctn – Toenail Factor (NDS•12.3.7) – applies to the W and Z values of toe-nailed connections (Ctn = 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. 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 Cg 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. 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.

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.

\[ W = 1380(G)^{2/5} DL_p \]

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. 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.

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.

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. The bending capacity of the washer should 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.

Any possible torsional load on a bolted connection (or any connection, for that matter) 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•Table 8.2 E. The yield equations (or general dowel equations) may also be used to conservatively determine the joint capacity.
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 pre-drilling. 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 Lp into the main member does not include the tapered portion at the point.

\[
W = 1800(G)^{3/5} D^{1/3} L_p^{0.3} \quad \text{unadjusted withdrawal design value (lb) for a lag screw}
\]

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 Ceg 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.

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. There may be alternative or improved design approaches. Similarly, some considerations regarding wood connection design are appropriate to address here.
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 past 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). 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 of the use of screws and deformed shank nails. However, the use of a standard geometric tributary area of the sheathing fastener and the wind loads, along with the NDS withdrawal values, 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 and ultimate withdrawal capacities.

It is interesting to note, however, that one study found that the lower-bound (i.e., 5th percentile) sheathing pull-off resistance was considerably higher than that predicted by the 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), and 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 have investigated the capacity of roof-to-wall (i.e., sloped rafter-to-top plate) connections using conventional toe-nailing and other enhancements (i.e., strapping,
brackets, gluing, etc.). Again, the primary concern is related to high wind conditions, such as those experienced during Hurricane Andrew and other extreme wind events.

First, as a matter of clarification, the toenail reduction factor Ctn does not apply to slant-nailing, such as those used for rafter-to-wall connections and floor-to-wall connections in conventional residential construction. Toe-nailing occurs when a nail is driven at an angle in a direction parallel to the 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 the 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 this is a generally reliable connection in most homes and similar structures built in the U.S., even a well-designed slant-nail connection used to attach roofs to walls is impractical in hurricane-prone regions or similar high-wind areas. In these conditions, a metal strap or bracket is preferable.

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 toenails) 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.
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-pound 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 Connect

The heel joint connection at the intersection of rafters and ceiling joists has long been considered one of the weaker connections in conventional wood roof framing. In fact, this highly stressed joint represents one of the significant reasons for 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, 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 3 to 5, relative to the ultimate capacity. Finally, the nail values should be adjusted for duration of the load (i.e., snow load duration factor of 1.15 to 1.25). 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. 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
Design of Concrete and Masonry Connections

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 are suitable for bolted wood connections to properly grouted masonry. 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.

Foundation Wall to Footing Connections

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.

Footing-to-wall connections for residential construction are constructed in any one of the following three ways (refer to Figure 5 for illustrations of the connections):

- no vertical reinforcement or key;
- key only; or
- dowel only.

Generally, no special connection is needed in non-hurricane-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.
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 (in pounds), $N$ is the dead load (psf), and $A$ is the bearing surface area (in square feet) 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 6 illustrates a footing with a key. Shear resistance developed by the key is computed in accordance with the equation below.

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.

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 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.

![FIGURE 7. Dowel Placement in Concrete Footings](image)

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 flatwork 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.

**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.

\[
\begin{align*}
V &= 6V_c \
V_c &= 6A_s f_c
\end{align*}
\]

**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.

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.

**Evaluating Problems with Fasteners**

The term "fasteners" typically refers to nails, screws, bolts, and sometimes anchors. Fasteners may directly join together two pieces of material, or the material may be held together by connectors that are, in turn, held in place by fasteners. A good deal of the
difficulty in evaluating fasteners is the fact that most home inspectors inspect existing structures, as opposed to homes under construction, so, by the time the inspector sees a fastener, there’s usually not much visible except its head. Certain problems affecting fasteners, such as corrosion, may be visible, but other problems may be apparent only to inspectors who understand their properties and those of the materials they join. In addition to becoming aware of visible issues, inspectors should understand some of the basics about fasteners that will help them spot less obvious problems.

There are many different types of fasteners. Let’s examine the most common types, as well as the problems they are subject to.

**Fastener Types and Their Applications**

Anchors are receptacle devices installed in very soft or very hard materials that alone wouldn’t hold or accept nails, screws or bolts well.

 undescribed image of fasteners

*This photo shows a metal connector called a joist hanger held in place by fasteners, some of which are correct for this particular connector, and some of which are not.*

In designing or specifying a fastener for a particular purpose, a designer has to take into consideration:

1. the types and extent of the force the fastener must resist;
2. the properties of the materials into which the fastener will be driven;
3. the various environmental elements that will act upon the fastener during its lifespan; and
4. the fastener’s lifespan requirements.

**STRUCTURAL FORCES**

Fasteners are designed to resist two structural forces: withdrawal and shear.

**Withdrawal**
The withdrawal force is parallel to the shaft of the fastener, called the shank. If you were to grab the head of a screw or nail with a pair of pliers and try to pull it straight out, the fastener would resist withdrawal.

One method used to help improve fastener resistance to withdrawal is to deform the fastener shank. This improves withdrawal resistance by increasing the friction that has to be overcome in order to withdraw the fastener.

Shank deformation takes a number of different forms. Adding threads to a fastener shaft to form a screw is one good way to achieve resistance.

**Drywall screws**

*#1 is a coarse-thread screw designed for use with wood studs.*

*#2 is a self-drilling, fine-thread screw designed for use with light-gauge steel studs.*

*#3 is a self-drilling, fine-thread screw designed for use with heavy-gauge steel studs.*

Coarse-thread screws can be installed faster but have lower withdrawal resistance than fine-thread screws.

Screws are more resistant to withdrawal than nails, but this does not mean that they can be substituted for nails for use with structural metal connectors. Fasteners used with metal connectors must be designed for use with each specific connector and approved by the connector manufacturer because connectors have load limitations that relate to a particular fastener’s properties and limitations.
Although there are structural screws on the market, most screws used with metal connectors are considered a defective installation. Structural screws are made from high-strength steel and heat-treated to further enhance their strength.

The SD wood screw is not approved for use with metal connectors. The structural screw is.

Head markings for Simpson screws

The structural screw in the diagram above is made by GRK. The CEE thread is designed to enlarge the hole in the uppermost of two pieces being joined so that they’ll be more tightly pulled together.
Ring-shank nail

Another method used to resist withdrawal is to roughen the nail shank by adding a series of rings. These are called ring-shank nails.

Roughened shank

Yet another method is to roughen the shank with coatings. In the photo above, compare the hot-dipped galvanized nail to the uncoated (bright) nail. Rough coatings are usually added to resist corrosion, but resistance to withdrawal is an additional advantage.

A spiral shank can also help resist withdrawal, although it’s one of the less common types of fasteners used in building construction.

Head-shank connection

In addition to the properties of the fastener shank, the strength of the connection of the head to the shank and the thickness of the head are important in resisting withdrawal.

Experiments

Inspectors should be aware of several experiments that have been conducted relative to withdrawal.

Gas & Wax

Before framing nails coated with vinyl became available in the mid-1970s, production framers working on large housing tracts in California found that uncoated nails took more of an effort to pound than they wanted to exert. So, to make nails easier to drive, they
would toss a bar of paraffin wax onto an open 50-pound box of 16d nails, pour on a little gasoline, and touch it off with a match. The wax would melt down through the box, making the nails much easier to drive, but lowering their withdrawal resistance dramatically. It also made it easier for the framers to hold onto a wood-handled hammer in hot weather. Baby powder was occasionally poured into the open box of nails for the same purpose, but it didn’t work as well as a lubricant.

**Shrunken, Roughened Shanks**

In the early 1990s, in an attempt to save money, some framing contractors substituted a slightly smaller nail with a roughened shank for the industry-standard, 16d hand-driven framing nail. However, this can lead to unexpected nail pull-out during construction with disastrous and potentially dangerous results.

Both gas and wax and the smaller substitute nails were used on many homes in California and a number of other places during the '90s, so if you see structural failures related to nail withdrawal, including head parts that are merely glued in place to give the appearance of being nailed, one of these issues or something similar may be the source of the problem. But there are some things you just won’t be able to spot.

**Shear**

Shear force is exerted perpendicular to the shank of a fastener. Fasteners that fasten metal connectors to wood are primarily designed to resist shear, although, in many applications, there will also be some withdrawal force involved, too. That's why fasteners for connectors also have minimum length requirements. The properties important to resisting shear are the strength of the alloy from which the fastener is made, its diameter, and the strength of the connection between the fastener shank and its head.

*A defective installation*

The fasteners used to connect the hanger to the wall pictured above are defective because the gold deck screws used are designed to resist withdrawal when holding deck planking to floor joists. They have inadequate shear strength to support the structural roof load. Also, because the drywall does not support the shank of the screw as adequately as wood does, the shear force is increased. Imagine that instead of resting against drywall, a ½-inch gap was left between the hanger and the framing. That’s almost the case. The roof of the garage next to this one collapsed under a snow load.
A similar defect with roofing nails

SCREW FAILURE

Screws fail in one of four ways:

1. Failure occurs through the shank. An example of this occurs when driving screws into a hard material. Screws often snap off just below the head. Deck screws may appear to be securely in place when, in fact, the shank has snapped. Although it looks secure, the head is detached from the shank and the screw has no holding power. You might find this problem by pushing on the materials the screw is designed to join to see if they move separately.

2. Stripping of the screw thread is common with a hard material and soft screw. The photo above was taken with an electron microscope and shows partially stripped threads.

3. Stripping of the internally threaded material is common with hard screws and soft material. Consider the example depicting a screw going through a marshmallow (see below).

4. The driver may strip the head. Slotted and Phillips-head screws strip more easily than screws with square or star drive profiles.
Screws used for fastening trim have heads smaller in diameter.

Fastener Lifespan

The lifespan of a fastener is related to its base material, which is usually carbon steel or one of a couple of different types of stainless steel. The type and thickness of the coating or plating will also affect the lifespan, with zinc being one of the most common coatings. The lifespan will also be affected by the properties of the materials that the fasteners are joining together and the environment in which the fastener is used.

Material Properties

Density

Dense materials provide a better anchoring substrate for resisting both withdrawal and shear.

To use an extreme example, oak holds fasteners more effectively than marshmallows.

Dense wood may need to have pilot holes pre-drilled to prevent it from splitting, especially near the ends. Dulling the end of a nail also helps prevent splitting, since the dull nail point crushes through wood fibers instead of wedging them apart as a sharp point does.

Some types of screws are designed to cut their own pilot holes. This screw is designed to fasten wood to steel and will cut its own pilot hole through steel.
Some materials, such as plastic-based composites used for decking, vary in density according to temperature and moisture content, so fastening requirements can vary from day to day. Extreme expansion and contraction have also made fastening these materials a challenge. According to an article in the September 2007 issue of *Building Products Digest* magazine, there were about 750,000 decks built in 2006 using plastic composite planking.

*A screw for fastening plastic-based composites.*

With as many as 80 manufacturers now offering composites of different formulations that are installed in widely differing climate zones, you may find decks with a large percentage of their fasteners that have spun out and have failed to hold the deck planking securely in place. Fastener manufacturers have been quick to provide solutions to these problems, and screws are now available for fastening composites used in a number of different environmental conditions.

*In this illustration, you can see how the tips of various screw types are designed to penetrate the materials that the screws were designed to fasten.*

**Thickness**

Materials that allow a fastener to remain in contact along its full length will provide more effective anchoring than a thinner material through which most of the fastener has penetrated and is no longer in contact.

*When thin materials, such as sheet metal, are joined together, screws with fully threaded shafts are used.*

When thicker materials, such as wood, are joined together, screws with a smooth section near the head allow the two pieces to be pulled tightly together.
This is a gold deck screw designed for fastening deck planking to joists.

**Chemical Reactions**

Metal fasteners can lose their load-bearing capacity when exposed to corrosive environments and materials. These include:

- preservative-treated wood;
- ocean salt air;
- fire-retardants;
- fertilizers;
- fumes; and
- acid rain.

Part of learning the inspection profession is learning not just about common conditions that can affect fasteners, but about conditions unique to the local region where you work that may affect fasteners.

**Preservative-Treated Wood**

Several types of water-borne preservatives were used in the past to increase wood’s resistance to attack by wood-destroying insects and decay fungi. Each type included chemicals that corrode some metals. Chemical formulas vary by manufacturer and region, and those formulas may change without warning. The level of retention of preservatives can vary by wood species and by the method used to treat the wood. Complicating the issue even further is that the industry is still evolving. So, although fastener manufacturers make recommendations about compatibility with their products, choosing the correct fastener or confirming that the right fastener has been used can be difficult, especially if all you can see is the fastener head in the spot of a flashlight in a dark basement or crawlspace.

Chromated copper arsenate (CCA) was used for many years, but its use has declined due to the inclusion of substantial amounts of arsenic as one of the treatment chemicals. U.S. EPA regulations in place since 2004 call for pressure-treatment chemicals to be arsenic-free. Generally, hot-dipped galvanized and stainless steel are the recommended fasteners for CCA.

The next generation of wood preservatives commonly used in buildings includes alkaline copperquat (ACQ), copper azole (Types A and B), as well as SBX/DOT (sodium borate) and zinc borate (for wood composites). The formulations for these products also vary. Although they don’t contain arsenic, some types contain chemicals that are more corrosive to fasteners than CCA.
The recommended fasteners for these include hot-dipped galvanized, stainless steel, or triple-coated zinc polymer materials. Carbon steel and aluminum fasteners should be avoided. Aluminum nails are not common in building and, in general, their use is limited to fastening aluminum flashing, so watch for bright nails used with treated lumber, and comment on this if you find them.

*A nail approved for use with treated lumber.*

Most stainless-steel fasteners are acceptable for use with pressure-treated wood. Testing has shown that Types 304 and 316 stainless steel perform well with CCA-C, ACQ-C, ACQ-D carbonate, CBA-A, and CA-B treated woods.

The large number of variables that affect the rate of corrosion of fasteners in contact with pressure-treated wood makes it impossible to provide an accurate, estimated long-term service life for these fasteners.

**PROTECTIVE COATINGS**

There are two basic types of corrosion-protection methods used to protect steel-based fasteners. Barrier coatings bond to the steel and serve as a shield between the steel and the corrosive elements in the environment. Sacrificial coatings often serve as a barrier coating. Additionally, because they’re lower on the anodic chart, they will corrode before steel so that even if the protective coating is damaged, exposing the steel, the sacrificial coating will corrode first, protecting the steel base metal.

**Bright**

Steel fasteners with no protective coating are called bright fasteners. Bright fasteners should be used in low-corrosive environments only. Even humid air will cause any exposed portions to eventually rust.

*A hot-dipped galvanized hanger nail above a bright hanger nail*
Fastener galvanization is the approved coating process most commonly used with pressure-treated lumber. Galvanization is the process of coating fasteners with zinc. The zinc coating acts as both a barrier coating, preventing corrosive agents from reaching the underlying steel base metal, and as a sacrificial coating, because zinc, as the more cathodic metal, will corrode before steel. There are several types of galvanization processes, including hot-dipped, electroplated and mechanically galvanized. The thicker the galvanized coating, the longer the expected long-term service life of the steel fastener.

Hot-dipped galvanized fasteners are used in regions where a maximum amount of protection is desired. To hot-dip galvanize steel fasteners, the steel is first cleaned, pickled, fluxed, and then dipped in a molten bath of zinc. The fasteners are allowed to cool prior to inspection and shipping. Some concrete anchors and metal connectors can also be hot-dip galvanized. Hot-dipped fasteners are manufactured to ASTM 153 standards.

Electro-galvanized fasteners are used in mild-weather conditions and in areas with low humidity. Electro-galvanization plates the nail in a zinc coating by using an electrical charge. The nails are submerged into an electrolytic solution and an electrical current coats them with a thin layer of zinc. However, after prolonged exposure to the elements, the thin layer of zinc oxidizes, leaving the fastener subject to normal rusting and staining.

Mechanical galvanizing is a process of providing a protective zinc coating over bare steel. The bare steel is cleaned and loaded into a tumbler containing non-metallic impact beads and zinc powder. As the tumbler is spun, the zinc powder mechanically adheres to the parts. The coating of mechanically coated nails is porous and brittle compared to electroplated and hot-dipped fasteners and is prone to flaking off.

Zinc-based coatings are one of the most common. Gold deck screws are simply zinc-plated screws dyed yellow to make them look like cadmium. Cadmium screws were used in the past because of their strength, but due to cadmium’s toxicity, it’s no longer used in fasteners that are typically used for building.
Vinyl-Coated Nails

A 16d vinyl-coated checker-head sinker, which is the industry standard.

Framing nails manufactured today are coated with vinyl, which acts as a lubricant when the fastener is being driven. It also provides a small amount of barrier protection against corrosion. It’s a common coating on hand-driven framing nails, such as 8d and 16d sinkers.

Resin-Coated Nails

Some nails are coated with a resin that acts as a lubricant for easier driving, and also as an adhesive. Driving the nail raises the temperature of the fastener enough to liquefy the resin. Once in place, the resin hardens and acts as an adhesive, bonding the shank to the wood fibers.

Phosphate-Coated Nails

Adding a thin coat of phosphate helps resist withdrawal and also provides a small measure of resistance to corrosion.

Galvanic Corrosion

Galvanic corrosion occurs when certain dissimilar metals come into contact with each other. Two conditions must exist for galvanic corrosion to take place:

1. There must be two dissimilar metals present.
2. There must be an electrically conductive path between the two metals, such as water.

This means that fasteners used with metal connectors or flashing should be made of the same metal as the connector. For instance, using stainless steel fasteners with galvanized steel connectors will likely lead to corrosion.

Cathodic Protection

A third type of basic protection from corrosion is called cathodic protection and consists of metals highly resistant to corrosion. Stainless steel Type 304 and especially Type 316 are the industry standards for fasteners used in building construction. Type 316 is recommended for salt environments, but you won’t be able to tell just by looking.
The photos above and below show stainless steel fasteners.

Copper nails resist corrosion well and are often used with copper trim and to attach slate roof tiles.

**Moisture Cycles**

Many commonly used construction materials, such as wood, expand and contract with changes in moisture content. This process is called moisture cycling. Over the long term, moisture cycling causes the holes around fasteners to enlarge, and when the fasteners used are nails, they eventually loosen in their holes and increasingly protrude as moist wood expands, gripping the nails and forcing them up and out of their holes slightly. As the wood dries, the holes enlarge, and the wood shrinks away from the nails.

As this cycle is repeated, nails can be raised above the wood surface significantly. Protruding nails are a common problem on decks with wood planking. This condition is also common on metal roofs with exposed fasteners, including screws.

**Fastener Sizes**

Screws are sized by number. This is a self-tapping, hex-head #10 zinc-plated screw.

Nails are sized by the “penny” shown as a “d.” This photo shows a 16d or 16-penny vinyl-coated checker-head sinker.
Masonry Anchors

Mechanical Anchors

*Masonry wedge anchor*

Wedge-type masonry anchors in sizes 3/8-inch, 1/2-inch and 5/8-inch, like those shown in the photo above, have a code stamped into the end that’s left exposed after the anchor is installed. Anchors 1½ inches are labeled A, 2-inch anchors are labeled B, 2½-inch anchors are labeled C, and so forth, with subsequent letters that correspond to length increasing by half-inch increments, as shown in the chart below.

*A wedge anchor code mark*

| Mark | A | B | C | D | E | F | G | H | I | J | K | L | M | N | O | P | Q | R | S | T | U | V | W | X | Y | Z |
| From | 1¼ | 2 | 2½ | 3 | 3¼ | 4 | 4¼ | 5 | 5¼ | 6 | 6¼ | 7 | 7½ | 8 | 8¼ | 9 | 9¼ | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 |
| Up To But Not Including | 2 | 2¼ | 3 | 3½ | 4 | 4½ | 5 | 5½ | 6 | 6½ | 7 | 7½ | 8 | 8½ | 9 | 9½ | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 |

*The code table*

Although you may see other fasteners with codes stamped into their heads like this stainless steel screw, codes are not standardized, so don’t assume you can tell the length by using the same chart that’s used for wedge anchors.

*Anchoring in cracked concrete has been a problem in the past, but a new type of wedge anchor is available for this use. It’s the anchor on the left in the photo above.*

Other Types of Concrete Anchors
Bear in mind that concrete anchors don’t work well in concrete masonry units (CMUs), commonly called concrete blocks, unless the cells are filled.

**Testing Anchor Connections**

Manufacturers of masonry anchors recommend confirming that the anchors are properly installed by testing them to the proper torque using a torque wrench. They do not recommend tapping anchor heads with hammers or tightening them with a socket wrench.

**Adhesive Anchors**

Adhesive anchors are usually threaded steel bar (commonly called all-thread) or re-bar that’s inserted into pre-drilled holes and held in place with an adhesive. The manufacturer’s instructions should be carefully followed for the anchors to attain their full strength. Holes should be drilled to the correct depth and diameter and then brushed and blown clean with compressed air.

Adhesive formulations can vary, resulting in widely differing performance characteristics among products with similar chemistry, including temperature-related performance. One problem with adhesive systems is known as “creep.” Some types of adhesives are designed to resist short-term loads only, such wind and seismic loads. When subjected to long-term loads, anchors will slowly pull loose.

In Boston in 2006, a portion of a suspended concrete ceiling system in a tunnel collapsed, killing one person. The adhesive anchors holding the ceiling in place, which were subjected to a long-term gravity load, pulled loose, resulting in the collapse. If you inspect structures
that may have adhesive anchors under long-term loads of some type, look carefully for signs of failure. One location where you might expect to see this in residential construction is where a concrete patio or porch has been retrofit to connect to a masonry foundation. Poor soil consolidation beneath the porch or patio slab that has resulted in settling may create a load on the anchors.

Concrete anchors have two test standards for creep, including the CC-ES AC 58, with the optional creep test. The other test is ICC-ES AC 308, which requires two sample tests taken at different temperatures.

**Drywall Anchors**

Different types of devices are available for anchoring screws into drywall. Some of the more common ones are shown below.

![Bolts](image1)

*1/8" Metal Hollow Wall Anchor, drill #6. Grip Range 3/8-1/2*

*10-12 x 1-1/2" Fluted Plastic Anchor*

*#8 Nylon Auger Type Anchor*

*13 oz. 16-14 x 1-1/2" Tapcon Plastic Anchor, drill #9.35*

**A typical zinc-plated, hex-head bolt.**

Two standards exist for grading bolts: the American National Standards Institute (ANSI) standard is for bolt strength. The International Standards Organization (ISO) standard is for both tensile and yield strength of the bolt.
A bolt graded by the ANSI standards is identified by the number of lines arranged around the head of the bolt.

- 0 lines = Grade 2 tensile strength
- 3 lines = Grade 5
- 5 lines = Grade 7
- 6 lines = Grade 8

A bolt graded by the ISO standard, shown in the photo below, uses two numbers on the head of the bolt. The first number indicates the tensile strength; the second number signifies the yield strength.

Most bolts used in residential building are Grade 5. Applications such as for steel wind frames may call for Grade 8 bolts, but, as an inspector, you’d need to see documentation showing that requirement. Seeking such confirmation exceeds InterNACHI’s Standards of Practice.

This is a carriage bolt. The square section beneath the head is designed to prevent the head from spinning as the nut is tightened.
This photo shows the difference in appearance between the head of a stainless-steel carriage bolt and a zinc carriage bolt.

This photo shows a plastic-lined lock-nut compared with a conventional hex-head nut.

LAG SCREWS

Lag screws are like heavy screws with hex heads.

Fastener Identification

Nails

This is a cut nail. As an older style, they’re not used much anymore, but you will see them used in older homes.

The photo above compares a galvanized finish nail above a stainless-steel siding nail.

Below are three views of nails commonly used for fastening metal connectors.
A masonry nail

This photo shows a TimberLok® screw designed for use with log and timber-framed homes.

The photos above and below show structural and wood screws manufactured by GRK.
Screws designed for use in masonry are often colored blue.

Nail Guns & Nails

The concern with fasteners installed with nail guns is over-driving the nails that are used to fasten structural floor, wall and roof panels made of materials such as plywood. In these cases, it’s important that the nails not be over-driven.

Over-driving nails (or driving them at an angle) reduces the effective thickness of the panel by breaking through its veneer.

Driver-Depth Adjustment Devices

Many newer guns have driver-depth adjustment devices built into the trigger mechanism. On nail guns that lack this device, the depth of the driver may be regulated by adjusting the air pressure at the compressor. This is less accurate, since the density of wood will vary.

A newer gun with a driver-depth adjustment device.
An older gun whose driver depth is regulated by air pressure.

**Gun Nails**

Framing nails for guns typically come in strips. Here are a few examples.

*Galvanized 12d*

*Bright, ring-shank 8d*

*Hot-dipped galvanized 6d*
Staples

A typical staple used in framing: 16-gauge galvanized 1½ x 7/16-inch

Home inspectors should be aware that fastener manufacturers do not give lifespans for their products because they vary too much based on where the fasteners are installed in a home, the materials in which they're installed, and the local climate and environment. However, inspectors can use the information presented here to make educated judgments about the materials they inspect.

Summary

The information in this course serves as a resource for both inspectors and designers who work with structural connections, and how these connections:

- transfer loads resisted by structural members and systems to other parts of the structure to form a continuous load path;
- secure non-structural components and equipment to the building; and
- fasten members in place during construction to resist temporary loads during installation.

Structural Connection Design Quiz Part 1

T/F: Connections transfer loads resisted by structural members and systems to other parts of the structure to form a continuous load path.

- True
- False

T/F: Structures are only as strong as their connections.

- True
- False
For joist to sill applications, ____ 8d nails are recommended.

- three
- two
- four
- five
- one

For header to joist applications, ____ 16d end nails are recommended.

- three
- one
- two
- four
- five

For connecting a stud to a top or bottom (sole) plate, ____ 16d end nails are recommended.

- two
- one
- three
- four

For connecting doubled studs, face-nailed 10d nails at 16 inches on center are to be ____ along the length of the stud.

- staggered
- in single-file alignment

The procedures for designing wood, concrete, and masonry connections cannot be found in:

- The 21st Century Structural Code and Design Manual
- Building Code Requirements for Structural Concrete
- The National Design Specification for Wood Construction
- Building Code Requirements for Masonry Structures

Mechanical Fasteners that are generally used for wood-framed house design and construction include the following except for:

- thread forming fasteners
- nails and spikes
- lag bolts (lag screws)
- bolts

Nail lengths and weights are denoted by penny weight, which is indicated by the letter:
In residential wood construction, bolt connections are typically limited to _______ connections unless the home is constructed in a high-hazard wind or seismic area, and hold-down brackets are required to transfer shear wall overturning forces.

- wood-to-concrete
- wood-to-wood
- wood-to-metal

_______ are bolted, nailed, or screwed to wall studs or posts and anchored to the construction below (concrete, masonry, or wood) to hold down the end of a member or assembly (i.e., shear wall).

- hold-down brackets
- joist hangers
- strap ties
- splice plates

**Structural Connection Design Quiz Part 2**

Structures are only as strong as their ______.

- connections
- finishes
- sheathing
- resistance

______ are bright or coated slender nails with a sinker head and diamond point.

- Sinker nails
- Common nails
- Power-driven nails
- Box nails

_____ are commonly supplied with ring shanks (i.e., annular threads) as a drywall nail.

- Cooler nails
- Sinker nails
- Common nails
- Box nails
bolts are often used for heavy connections and to secure wood to other materials, such as steel or concrete.

- bolts
- nails
- fasteners
- spikes

bolts, unlike nails, are installed in pre-drilled holes.

- bolts, nails
- nails, bolts
- wood, metal
- metal, wood

The moisture condition of the wood is also critical to long-term connection performance, particularly for nails in withdrawal.

- moisture
- heat

The moisture condition of the wood is also critical to long-term connection performance, particularly for nails in withdrawal.

- moisture
- heat
- grain

The design strength of nails is greater when a nail is driven into the side rather than the end grain of a member.

- greater
- weaker
- equal

The diameter of a common nail is larger than that of sinkers and box nails of the same length.

- larger
- shorter
- equal to

Generally, no special connection is needed in non-hurricane-prone or low- to moderate-hazard seismic areas.

- no special connection
• a special connection
• a strong connection
• a simple connection

A _______ key is commonly used to interlock foundation walls to footings.

• concrete
• metal
• wood
• steel

If development length ______ the footing thickness, the dowel must be in the form of a hook, which is rarely required in residential construction.

• exceeds
• is shorter than
• is equal to

Therefore, this guide suggests a minimum embedment length of __ to ___ inches for footing dowels, when necessary, in residential construction applications.

• 6, 8
• 8, 10
• 12, 14
• 4, 6

The term "________" typically refers to nails, screws, bolts, and sometimes anchors.

• fasteners
• deck screw
• hanger nail
• sliding nail

The term "________" typically refers to nails, screws, bolts, and sometimes anchors.

• fasteners
• deck screw
• hanger nail
• sliding nail

The withdrawal force is parallel to the shaft of the fastener, called the ______.
Coarse-thread screws can be installed faster but have ______ withdrawal resistance than fine-thread screws.

- lower
- greater

Another method used to resist withdrawal is to roughen the nail shank by adding a series of ______.

- rings
- screws
- bolts
- connectors

______ are used in lieu of foundation anchor bolts.

- Sill anchors
- Joist hangers
- Rafter clips
- Strap ties

______ are used to attach single or multiple joists to the side of girders or header joists.

- Joist hangers
- Sill anchors
- Rafter clips
- Strap ties

______ are flat plates with pre-punched holes for fasteners to transfer shear or tension forces across a joint.

- Splice plates
- Joist hangers
- Rafter clips
- Strap ties

______ 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.

- Anchor bolts
- Joist hangers
- Rafter clips
- Strap ties
_____ are pre-punched straps or coils of strapping that are used for a variety of connections to transfer tension loads.

- Strap ties
- Epoxy-set anchors
- Splice plates
- Sill anchors

_____ are straps or brackets that connect roof framing members to wall framing to resist roof uplift loads associated with high-wind conditions.

- Rafter clips
- Strap ties
- Splice plates
- Sill anchors

_____ are brackets that are bolted, nailed, or screwed to wall studs or posts and anchored to the construction below (concrete, masonry or wood) to hold down the end of a member or assembly (i.e., shear wall).

- Hold-down brackets
- Rafter clips
- Splice plates
- Anchor bolts

The heel joint connection at the intersection of rafters and ceiling joists has long been considered one of the _____ connections in conventional wood roof framing.

- weaker
- stronger

A wide but thin washer _____ evenly distribute the bearing force to the surrounding wood.

- will not
- will