CHAPTER 5

Design of Wood Framing

5.1 General

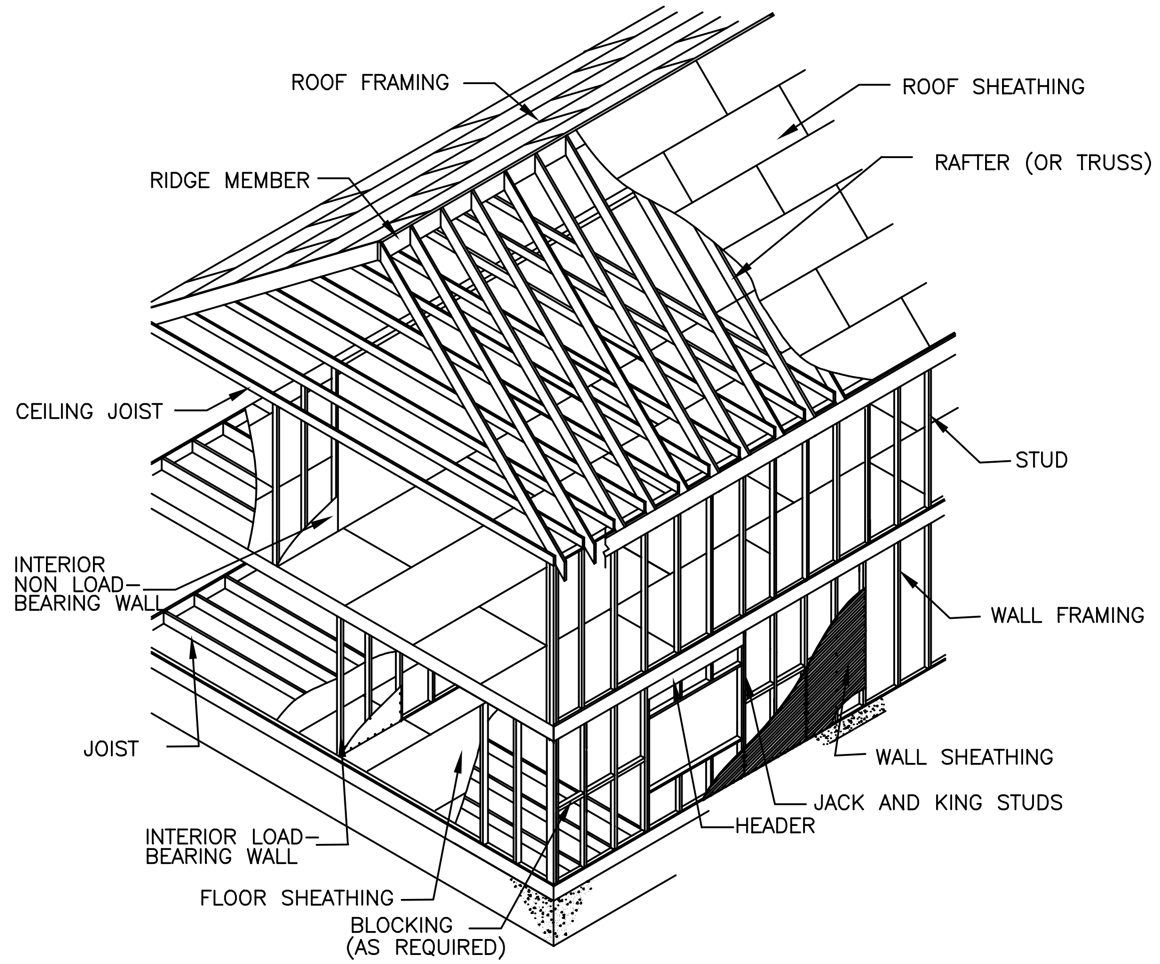
This chapter addresses elements of above-grade structural systems in residential construction. As discussed in Chapter 1, the residential construction material most commonly used above grade in the United States is light-frame wood; therefore, this chapter focuses on structural design that specifies standard dimension lumber and structural wood panels (i.e., plywood and oriented strand board sheathing). Design of the lateral force resisting system (i.e., shearwalls and diaphragms) must be approached from a system design perspective and is addressed in Chapter 6. Connections are addressed in Chapter 7, and their importance relative to the overall performance of wood-framed construction cannot be overemphasized. The basic components and assemblies of a conventional wood frame home are shown in Figure 5.1; the reader is referred to Chapter 1 for more detailed references to house framing and related construction details.

Many elements of a home work together as a system to resist lateral and axial forces imposed on the above-grade structure and transfer them to the foundation. The above-grade structure also helps resist lateral soil loads on foundation walls through connection of floor systems to foundations. Therefore, the issue of system performance is most pronounced in the above-grade assemblies of light-frame homes. Within the context of simple engineering approaches that are familiar to designers, system-based design principles are addressed in this Chapter.

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

- floors;
- walls; and
- roofs.

FIGURE 5.1 ***Components and Assemblies of a Conventional Wood-Framed Home***



Each system can be complex to design as a whole; therefore, simple analysis usually focuses on the individual elements that constitute the system. In some cases, "system effects" may be considered in simplified form and applied to the design of certain elements that constitute specifically defined systems. Structural elements that make up a residential structural system include:

- bending members;
- columns;
- combined bending and axial loaded members;
- sheathing (i.e., diaphragm); and
- connections.

The principal method of design for wood-framed construction has historically been allowable stress design (ASD). This chapter uses the most current version of the ASD method (AF&PA, 1997), although the load resistance factored design method (LRFD) is now available as an alternative (AF&PA, 1996a). The ASD method is detailed in the *National Design Specification for Wood Construction* (NDS) and its supplement (NDS-S). The designer is encouraged to obtain the NDS commentary to develop a better understanding of the rationale and substantiation for the NDS (AF&PA, 1999).

This chapter looks at the NDS equations in general and includes design examples that detail the appropriate use of the equations for specific structural elements or systems in light, wood-framed construction. The discussion focuses primarily on framing with traditional dimension lumber but gives some consideration to common engineered wood products. Other wood framing methods, such as post-and-beam construction, are not explicitly addressed in this chapter, although much of the information is relevant. However, system considerations and system factors presented in this chapter are only relevant to light, wood-framed construction using dimension lumber.

Regardless of the type of structural element to analyze, the designer must first determine nominal design loads. The loads acting on a framing member or system are usually calculated in accordance with the applicable provisions of the locally approved building code and engineering standards. The nominal design loads and load combinations used in this chapter follow the recommendations in Chapter 3 for residential design.

While prescriptive design tables (i.e., span tables) and similar design aids commonly used in residential applications are not included herein, the designer may save considerable effort by consulting such resources. Most local, state, or national model building codes such as the *One- and Two-Family Dwelling Code* (ICC, 1998) contain prescriptive design and construction provisions for conventional residential construction. Similar prescriptive design aids and efficient framing practices can be found in *Cost-Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994). For high wind conditions, prescriptive guidelines for design and construction may be found in the *Wood Frame Construction Manual for One- and Two-Family Dwellings* (AFPA, 1996b). The designer is also encouraged to obtain design data on a variety of proprietary engineered wood products that are suitable for many special design needs in residential construction. However, these materials generally should not be viewed as simple "one-to-one" substitutes for conventional wood framing and any special design and construction requirements should be carefully considered in accordance with the manufacturer's recommendation or applicable code evaluation reports.

5.2 Material Properties

It is essential that a residential designer specifying wood materials appreciate the natural characteristics of wood and their effect on the engineering properties of lumber. A brief discussion of the properties of lumber and structural wood panels follows.

5.2.1 Lumber

General

As with all materials, the designer must consider wood's strengths and weaknesses. A comprehensive source of technical information on wood characteristics is the *Wood Engineering Handbook, Second Edition* (Forest Products Laboratory, 1990). For the most part, the knowledge embodied in the handbook is reflected in the provisions of the NDS and the NDS Supplement (NDS-S) design data; however, many aspects of wood design require good judgment.

Wood is a natural material that, as a structural material, demonstrates unique and complex characteristics. Wood's structural properties can be traced back to the material's natural composition. Foremost, wood is a nonhomogeneous, non-isotropic material, and thus exhibits different structural properties depending on the orientation of stresses relative to the grain of the wood. The grain is produced by a tree's annual growth rings, which determine the properties of wood along three orientations: tangential, radial, and longitudinal.

Given that lumber is cut from logs in the longitudinal direction, the grain is parallel to the length of a lumber member. Depending on where the lumber is cut relative to the center of a log (i.e., tangential versus radial), properties vary across the width and thickness of an individual member.

Wood Species

Structural lumber can be manufactured from a variety of wood species; however, the various species used in a given locality are a function of the economy, regional availability, and required strength properties. A wood species is classified as either hardwood or softwood. *Hardwoods* are broad-leafed deciduous trees while s*oftwoods* (i.e., conifers) are trees with needle-like leaves and are generally evergreen.

Most structural lumber is manufactured from softwoods because of the trees' faster growth rate, availability, and workability (i.e., ease of cutting, nailing, etc.). A wood species is further classified into groups or combinations as defined in the NDS. Species within a group have similar properties and are subject to the same grading rules. Douglas Fir-Larch, Southern Yellow Pine, Hem-Fir, and Spruce-Pine-Fir are species groups that are widely used in residential applications in the United States.

Lumber Sizes

Wood members are referred to by nominal sizes (e.g., 2x4); however, true dimensions are somewhat less. The difference occurs during the dressing stage of the lumber process, when each surface of the member is planed to its final dressed dimension after shrinkage has occurred as a result of the drying or "seasoning" process. Generally, there is a 1/4- to 3/4-inch difference between the nominal and dressed sizes of "dry" sawn lumber (refer to NDS-S Table 1B for specific dimensions). For example, a 2x4 is actually 1.5 inches by 3.5 inches, a 2x10 is 1.5

inches by 9.25 inches, and a 1x4 is 3/4-inch by 3.5 inches. This guide uses nominal member size, but it is important to note that the designer must apply the actual dimensions of the lumber when analyzing structural performance or detailing construction dimensions.

Based on the expected application, the tabulated values in the NDS are classified by the species of wood as well as by the nominal size of a member. Typical NDS classifications follow:

- *Boards* are less than 2 inches thick.
- *Dimension lumber* is a minimum of 2 inches wide and 2 to 4 inches thick.
- *Beams and stringers* are a minimum of 5 inches thick, with the width at least 2 inches greater than the thickness dimension.
- *Posts and timbers* are a minimum of 5 inches thick, and the width does not exceed the thickness by more than 2 inches.
- *Decking* is 2 to 4 inches thick and loaded in the weak axis of bending for a roof, floor, or wall surface.

Most wood used in light-frame residential construction takes the form of dimension lumber.

Lumber Grades

Lumber is graded in accordance with standardized grading rules that consider the effect of natural growth characteristics and “defects,” such as knots and angle of grain, on the member’s structural properties. Growth characteristics reduce the overall strength of the member relative to a “perfect,” clear-grained member without any natural defects. Most lumber is visually graded, although it can also be machine stress-rated or machine evaluated.

Visually graded lumber is graded by an individual who examines the wood member at the mill in accordance with an approved agency’s grading rules. The grader separates wood members into the appropriate grade classes. Typical visual grading classes in order of decreasing strength properties are Select Structural, No. 1, No. 2, Stud, etc. Refer to the NDS Supplement (NDS-S) for more information on grades of different species of lumber. The designer should consult a lumber supplier or contractor regarding locally available lumber species and grades.

Machine stress rated (MSR) and *machine evaluated lumber* (MEL) is subjected to nondestructive testing of each piece. The wood member is then marked with the appropriate grade stamp, which includes the allowable bending stress (F_b) and the modulus of elasticity (E). This grading method yields lumber with more consistent structural properties than visual grading only.

While grading rules vary among grading agencies, the U.S. Department of Commerce has set forth minimums for voluntary adoption by the recognized lumber grading agencies. For more information regarding grading rules, refer to *American Softwood Lumber Voluntary Product Standard* (USDOC PS-20), which is maintained by the National Institute for Standards and Technology (NIST, 1994). NDS-S lists approved grading agencies and roles.

Moisture Content

Wood properties and dimensions change with moisture content (MC). Living wood contains a considerable amount of free and bound water. Free water is contained between the wood cells and is the first water to be driven off in the drying process. Its loss affects neither volume nor structural properties. Bound water is contained within the wood cells and accounts for most of the moisture under 30 percent; its loss results in changes in both volume (i.e., shrinkage) and structural properties. The strength of wood peaks at about 10 to 15 percent MC.

Given that wood generally has an MC of more than 30 percent when cut and may dry to an equilibrium moisture content (EMC) of 8 to 10 percent in protected environment, it should be sufficiently dried or seasoned before installation. Proper drying and storage of lumber minimizes problems associated with lumber shrinkage and warping. A minimum recommendation calls for using “surface dry” lumber with a maximum 19 percent MC. In uses where shrinkage is critical, specifications may call for “KD-15,” which is kiln-dried lumber with a maximum moisture content of 15 percent. The tabulated design values in the NDS are based on a moisture content of 19 percent for dimension lumber.

The designer should plan for the vertical movement that may occur in a structure as a result of shrinkage. For more complicated structural details that call for various types of materials and systems, the designer might have to account for differential shrinkage by isolating members that will shrink from those that will maintain dimensional stability. The designer should also detail the structure such that shrinkage is as uniform as possible, thereby minimizing shrinkage effects on finish surfaces. When practical, details that minimize the amount of wood transferring loads perpendicular-to-grain are preferable.

Shrink and swell can be estimated in accordance with Section 5.3.2 for the width and thickness of wood members (i.e., tangentially and radially with respect to annual rings). Shrinkage in the longitudinal direction of a wood member (i.e., parallel to grain) is negligible.

Durability

Moisture is a primary factor affecting the durability of lumber. Fungi, which feed on wood cells, require moisture, air, and favorable temperatures to survive. When wood is subject to moisture levels above 20 percent and other favorable conditions, decay begins to set in. Therefore, it is important to protect wood materials from moisture, by:

- limiting end use (e.g., specifying interior applications or isolating lumber from ground contact);
- using a weather barrier (e.g., siding, roofing, building wrap, flashing, etc.);
- applying a protective coating (e.g., paint, water repellent, etc.);

- installing roof overhangs and gutters; and
- specifying preservative-treated or naturally decay-resistant wood.

For homes, an exterior weather barrier (e.g., roofing and siding) protects most structural wood. However, improper detailing can lead to moisture intrusion and decay. Problems are commonly associated with improper or missing flashing and undue reliance on caulking to prevent moisture intrusion. For additional information and guidance on improving the durability of wood in buildings, refer to *Prevention and Control of Decay in Homes* (HUD, 1978).

Wood members that are in ground contact should be preservative treated. The most common lumber treatment is CCA (copper-chromium-arsenate), which should be used for applications such as sill plates located near the ground or for exterior decks. It is important to specify the correct level of treatment (0.4 pcf retention for nonground-contact exterior exposure and 0.6 pcf for ground contact).

Termites and other wood-destroying insects (e.g., carpenter ants, boring beetles, etc.) attack wood materials. Some practical solutions include: the chemical treatment of soil; the installation of physical barriers (e.g., termite shields); and the specification of treated lumber.

Termites are a special problem in warmer climates, although they also plague many other areas of the United States. The most common termites are “subterranean” termites that nest in the ground and enter wood that is near or in contact with damp soil. They gain access to above-grade wood through cracks in the foundation or through shelter tubes (i.e., mud tunnels) on the surface of foundation walls. Since the presence of termites lends itself to be visual to detection, wood-framed homes require periodic inspection for signs of termites.

5.2.2 Structural Wood Panels

Historically, boards were used for roof, floor, and wall sheathing; in the last 30 years, however, structural wood panel products have come to dominate the sheathing market. Structural wood panel products are more economical and efficient and can be stronger than traditional board sheathing. Structural wood panel products primarily include plywood and oriented strand board (OSB).

Plywood is manufactured from wood veneers glued together under high temperature and pressure. Each veneer or ply is placed with its grain perpendicular to the grain of the previous layer. The outer layers are placed with their grain parallel to the longer dimension of the panel. Thus, plywood is stronger in bending along the long direction and should be placed with the long dimension spanning floor and roof framing members. The number of plies typically ranges from 3 to 5. Oriented strand board is manufactured from thin wood strands glued together under high temperature and pressure. The strands are layered and oriented to produce strength properties similar to plywood; therefore, the material is used for the same applications as plywood.

The designer should specify the grade and span rating of structural wood panels to meet the required application and loading condition (i.e., roof, wall or floor). The most common panel size is 4x8 feet panels, with thicknesses typically ranging from 3/8-inch to more than 1 inch. Panels can be ordered in longer lengths for special applications.

Plywood is performance-rated according to the provisions of USDOC PS-1 for industrial and construction plywood (NIST, 1995). OSB products are performance-rated according to the provisions of USDOC PS-2 (NIST, 1992). However, these standards are voluntary and not all wood-based panel products are rated accordingly. The APA–Engineered Wood Association's (formerly American Plywood Association) rating system for structural wood panel sheathing products and those used by other structural panel trademarking organizations are based on the U.S. Department of Commerce voluntary product standards.

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

GradeA: The highest-quality veneer grade, which is intended for cabinet or furniture use.
Grade B: A high-quality veneer grade, which is intended for cabinet or furniture use with all defects repaired.
Grade C: The minimum veneer grade, which is intended for exterior use.
Grade D: The lowest-quality veneer grade, which is intended for interior use or where protected from exposure to weather.

The wood strands or veneer layers used in wood structural panels are bonded with adhesives and they vary in moisture resistance. Therefore, wood structural panels are also classified with respect to end-use exposure as follows:

- *Exterior* panels are designed for applications with permanent exposure to the weather or moisture.
- *Exposure 1* panels are designed for applications where temporary exposure to the weather due to construction sequence may be expected.
- *Exposure 2* panels are designed for applications with a potential for high humidity or wetting but are generally protected during construction.
- *Interior* panels are designed for interior applications only.

Typical span ratings for structural wood panels specify either the maximum allowable center-to-center spacing of supports (e.g., 24 inches on center for roof, floor, or wall) or two numbers separated by a slash to designate the allowable center-to-center spacing of roof and floor supports, respectively (e.g., 48/24). Even though the second rating method does not specifically indicate wall stud spacing, the panels may also be used for wall sheathing. The *Design and Construction Guide: Residential and Commercial* provides a correlation between roof/floor ratings and allowable wall support spacing (APA, 1998a). The *Load-Span Tables for APA Structural-Use Panels* (APA, 1999) provided span ratings for various standard and nonstandard loading conditions and deflection limits.

5.2.3 Lumber Design Values

The NDS-S provides tabulated design stress values for bending, tension parallel to grain, shear parallel to grain, compression parallel and perpendicular to grain, and modulus of elasticity. In particular, the 1997 edition of the NDS includes the most up-to-date design values based on test results from an eight-year full-scale testing program that uses lumber samples from mills across the United States and Canada.

Characteristic structural properties for use in allowable stress design (ASTM D1990) and load and resistance factor design (ASTM D5457) are used to establish design values (ASTM, 1998a; ASTM, 1998b). Test data collected in accordance with the applicable standards determine a characteristic strength value for each grade and species of lumber. The value is usually the mean (average) or fifth percentile test value. The fifth percentile represents the value that 95 percent of the sampled members exceeded. In ASD, characteristic structural values are multiplied by the reduction factors in Table 5.1. The reduction factors are implicit in the allowable values published in the NDS-S for standardized conditions. The reduction factor normalizes the lumber properties to a standard set of conditions related to load duration, moisture content, and other factors. It also includes a safety adjustment if applicable to the particular limit state (i.e., ultimate capacity). Therefore, for specific design conditions that differ from the standard basis, design property values should be adjusted as described in Section 5.2.4.

The reduction factors in Table 5.1 are derived as follows as reported in ASTM D2915 (ASTM, 1997):

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

5.2.4 Adjustment Factors

The allowable values published in the NDS-S are determined for a standard set of conditions. Yet, given the many variations in the characteristics of wood that affect the material's structural properties, several adjustment factors are available to modify the published values. For efficient design, it is important to use the appropriate adjustments for conditions that vary from those used to derive the standard design values. Table 5.2 presents adjustment factors that apply to different structural properties of wood. The following sections briefly discuss the adjustment factors most commonly used in residential applications. For information on other adjustment factors, refer to the NDS, NDS-S, and the NDS commentary.

TABLE 5.1 Design Properties and Associated Reduction Factors for ASD

Stress Property	Reduction Factor	Basis of Estimated Characteristic Value from Test Data	Limit State	ASTM Designation
Extreme fiber stress in bending, F_b	$\frac{1}{2.1}$	Fifth percentile	Ultimate capacity	D1990
Tension parallel to grain, F_t	$\frac{1}{2.1}$	Fifth percentile	Ultimate capacity	D1990
Shear parallel to grain, F_v	$\frac{1}{4.1}$	Fifth percentile	Ultimate capacity	D245
Compression parallel to grain, F_c	$\frac{1}{1.9}$	Fifth percentile	Ultimate capacity	D1990
Compression perpendicular to grain, $F_{c\perp}$	$\frac{1}{1.5}$	Mean	0.04” deflection[1]	D245
Modulus of elasticity, E	$\frac{1}{1.0}$	Mean	Proportional limit[2]	D1990

Sources: ASTM, 1998a; ASTM, 1998c.
Notes:
[1]The characteristic design value for $F_{c\perp}$ is controlled by a deformation limit state. In fact, the lumber will densify and carry an increasing load as it is compressed.
[2]The proportional limit of wood load-deformation behavior is not clearly defined because it is nonlinear. Therefore, designation of a proportional limit is subject to variations in interpretation of test data.

TABLE 5.2 Adjustment Factor Applicability to Design Values for Wood

Design Properties[1]	Adjustment Factor[2]														
	C_D	C_r	C_H	C_F	C_P	C_L	C_M	C_{fu}	C_b	C_T	C_V	C_t	C_i	C_c	C_f
F_b	✔	✔		✔		✔	✔	✔			✔	✔	✔	✔	✔
F_t	✔			✔			✔					✔	✔		
F_v	✔		✔				✔					✔	✔		
$F_{c\perp}$							✔		✔			✔	✔		
F_c	✔			✔	✔		✔					✔	✔		
E							✔			✔		✔	✔		

Source: Based on NDS•2.3 (AF&PA, 1997).
Notes:
[1]Basic or unadjusted values for design properties of wood are found in NDS-S. See Table 5.1 for definitions of design properties.
[2]Shaded cells represent factors most commonly used in residential applications; other factors may apply to special conditions.

Key to Adjustment Factors:

- C_D, Load Duration Factor. Applies when loads are other than "normal" 10-year duration (see Section 5.2.4.1 and NDS•2.3.2).

- C_r, Repetitive Member Factor. Applies to bending members in assemblies with multiple members spaced at maximum 24 inches on center (see Section 5.2.4.2 and NDS•4.3.4).

- C_H, Horizontal Shear Factor. Applies to individual or multiple members with regard to horizontal, parallel-to-grain splitting (see Section 5.2.4.3 and NDS-S).
- C_F, Size Factor. Applies to member sizes/grades other than "standard" test specimens, but does not apply to Southern Yellow Pine (see Section 5.2.4.4 and NDS-S).
- C_P, Column Stability Factor. Applies to lateral support condition of compression members (see Section 5.2.4.5 and NDS•3.7.1).
- C_L, Beam Stability Factor. Applies to bending members not subject to continuous lateral support on the compression edge (see Section 5.2.4.6 and NDS•3.3.3).
- C_M, Wet Service Factor. Applies where the moisture content is expected to exceed 19 percent for extended periods (see NDS-S).
- C_{fu}, Flat Use Factor. Applies where dimension lumber 2 to 4 inches thick is subject to a bending load in its weak axis direction (see NDS-S).
- C_b, Bearing Area Factor. Applies to members with bearing less than 6 inches and not nearer than 3 inches from the members' ends (see NDS•2.3.10).
- C_T, Buckling Stiffness Factor. Applies only to maximum 2x4 dimension lumber in the top chord of wood trusses that are subjected to combined flexure and axial compression (see NDS•4.4.3).
- C_V, Volume Factor. Applies to glulam bending members loaded perpendicular to the wide face of the laminations in strong axis bending (see NDS•5.3.2).
- C_t, Temperature Factor. Applies where temperatures exceed 100^{o}F for long periods; not normally required when wood members are subjected to intermittent higher temperatures such as in roof structures (see NDS•2.4.3 and NDS•Appendix C).
- C_i, Incising Factor. Applies where structural sawn lumber is incised to increase penetration of preservatives with small incisions cut parallel to the grain (see NDS•2.3.11).
- C_c, Curvature Factor. Applies only to curved portions of glued laminated bending members (see NDS•5.3.4).
- C_f, Form Factor. Applies where bending members are either round or square with diagonal loading (see NDS•2.3.8).

5.2.4.1 Load Duration Factor (C_D)

Lumber strength is affected by the cumulative duration of maximum variable loads experienced during the life of the structure. In other words, strength is affected by both the load intensity and its duration (i.e., the load history). Because of its natural composition, wood is better able to resist higher short-term loads (i.e., transient live loads or impact loads) than long-term loads (i.e., dead loads and sustained live loads). Under impact loading, wood can resist about twice as much stress as the standard 10-year load duration (i.e., "normal duration") to which wood bending stress properties are normalized in the NDS.

When other loads with different duration characteristics are considered, it is necessary to modify certain tabulated stresses by a load duration factor (C_D) as shown in Table 5.3. Values of the load duration factor, C_D, for various load types are based on the total accumulated time effects of a given type of load during the useful life of a structure. C_D increases with decreasing load duration.

Where more than one load type is specified in a design analysis, the load duration factor associated with the shortest duration load is applied to the entire combination of loads. For example, for the load combination, *Dead Load + Snow Load + Wind Load*, the load duration factor, C_D, is equal to 1.6.

TABLE 5.3 ***Recommended Load Duration Factors for ASD***

Load Type	Load Duration	Recommended C_D Value
Permanent (dead load)	Lifetime	0.9
Normal	Ten years	1.0
Occupancy (live load)[1]	Ten years to seven days	1.0 to 1.25
Snow[2]	One month to seven days	1.15 to 1.25
Temporary construction	Seven days	1.25
Wind and seismic[3]	Ten minutes to one minute	1.6 to 1.8
Impact	One second	2.0

Source: Based on NDS•2.3.2 and NDS•Appendix B (AF&PA, 1997).
Notes:
[1]The NDS uses a live load duration of ten years (C_D = 1.0). The factor of 1.25 is consistent with the time effect factor for live load used in the new wood LRFD provisions (AF&AP, 1996a).
[2]The NDS uses a snow load duration of one month (C_D = 1.15). The factor of 1.25 is consistent with the time effect factor for snow load used in the new wood LRFD provisions (AF&PA, 1996a).
[3]The NDS uses a wind and seismic load duration of ten minutes (C_D = 1.6). The factor may be as high as 1.8 for earthquake loads which generally have a duration of less than 1 minute with a much shorter duration for ground motions in the design level range.

5.2.4.2 Repetitive Member Factor (C_r)

When three or more parallel dimension lumber members are spaced a maximum of 24 inches on center and connected with structural sheathing, they comprise a structural “system” with more bending capacity than the sum of the single members acting individually. Therefore, most elements in a house structure benefit from an adjustment for the system strength effects inherent in repetitive members.

The tabulated design values given in the NDS are based on single members; thus, an increase in allowable stress is permitted in order to account for repetitive members. While the NDS recommends a repetitive member factor of 1.15 or a 15 percent increase in bending strength, system assembly tests have demonstrated that the NDS repetitive member factor is conservative for certain conditions. In fact, test results from several studies support the range of repetitive member factors shown in Table 5.4 for certain design applications. As shown in Table 5.2, the adjustment factor applies only to extreme fiber in bending, F_b. Later sections of Chapter 5 cover other system adjustments related to concentrated loads, header framing assemblies, and deflection (stiffness) considerations.

TABLE 5.4 ***Recommended Repetitive Member Factors for Dimension Lumber Used in Framing Systems***[1,2]

Application	Recommended C_r Value	References
Two adjacent members sharing load[3]	1.1 to 1.2	AF&PA,1996b HUD, 1999
Three adjacent members sharing load[3]	1.2 to 1.3	ASAE, 1997
Four or more adjacent members sharing load[3]	1.3 to 1.4	ASAE, 1997
Three or more members spaced not more than 24 inches on center with suitable surfacing to distribute loads to adjacent members (i.e., decking, panels, boards, etc.)[4]	1.15	NDS
Wall framing (studs) of three or more members spaced not more than 24 inches on center with minimum 3/8-inch-thick wood structural panel sheathing on one side and 1/2-inch thick gypsum board on the other side[5]	1.5–2x4 or smaller 1.35–2x6 1.25–2x8 1.2–2x10	AF&PA, 1996b SBCCI, 1999 Polensek, 1975

Notes:
[1]NDS recommends a C_r value of 1.15 only as shown in the table. The other values in the table were obtained from various codes, standards, and research reports as indicated.
[2]Dimension lumber bending members are to be parallel in orientation to each other, continuous (i.e., not spliced), and of the same species, grade, and size. The applicable sizes of dimension lumber range from 2x4 to 2x12.
[3]C_r values are given as a range and are applicable to built-up columns and beams formed of continuous members with the strong-axis of all members oriented identically. In general, a larger value of C_r should be used for dimension lumber materials that have a greater variability in strength (i.e., the more variability in strength of individual members the greater the benefit realized in forming a built-up member relative to the individual member strength). For example, a two-ply built-up member of No. 2 grade (visually graded) dimension lumber may qualify for use of a C_r value of 1.2 whereas a two-ply member of No. 1 dense or mechanically graded lumber may qualify for a C_r value of 1.1. The individual members should be adequately attached to one another or the load introduced to the built-up member such that the individual members act as a unit (i.e., all members deflect equally) in resisting the bending load. For built-up bending members with non-continuous plys (i.e., splices), refer to ASAE EP 559 (ASAE, 1997). For built-up columns subject to weak axis bending load or buckling, refer to ASAE EP 559 and NDS•15.3.
[4]Refer to NDS•4.3.4 and the NDS *Commentary* for additional guidance on the use of the 1.15 repetitive member factor.
[5]The C_r values are based on wood structural panel attachment to wall framing using 8d common nails spaced at 12 inches on center. For fasteners of a smaller diameter, multiply the C_r values by the ratio of the nail diameter to that of an 8d common nail (0.131 inch diameter). The reduction factor applied to C_r need not be less than 0.75 and the resulting value of C_r should not be adjusted to less than 1.15. Doubling the nailing (i.e., decreasing the fastener spacing by one-half) can increase the C_r value by 16 percent (Polensek, 1975).

With the exception of the 1.15 repetitive member factor, the NDS does not currently recognize the values in Table 5.4. Therefore, the values in Table 5.4 are provided for use by the designer as an “alternative” method based on various sources of technical information including certain standards, code recognized guidelines, and research studies. For more information on system effects, consult the following sample of references:

"Structural Performance of Light-Frame Truss-Roof Assemblies" (Wolfe, 1996).

“Performance of Light-Frame Redundant Assemblies” (Wolfe, 1990).

“Reliability of Wood Systems Subjected to Stochastic Live Loads” (Rosowsky and Ellingwood, 1992).

“System Effects in Wood Assemblies” (Douglas and Line, 1996).

Design Requirements and Bending Properties for Mechanically Laminated Columns (EP 559) (ASAE, 1997).

Rational Design Procedure for Wood Stud Walls Under Bending and Compression Loads (Polensek, 1975).

Stress and Deflection Reduction in 2x4 Studs Spaced 24 Inches On Center Due to the Addition of Interior and Exterior Surfacings (NAHBRF, 1974).

Structural Reliability Analysis of Wood Load Sharing Systems (Bonnicksen and Suddarth, 1965).

System Performance of Wood Header Assemblies (HUD, 1999).

Wall & Floor Systems: Design and Performance of Light-Frame Structures (FPRS, 1983).

5.2.4.3 Horizontal Shear Factor (C_H)

Given that lumber does not dry uniformly, it is subject to warping, checking, and splitting, all of which reduce the strength of a member. The horizontal stress values in the NDS-S conservatively account for any checks and splits that may form during the seasoning process and, as in the worst-case values, assume substantial horizontal splits in all wood members. Although a horizontal split may occur in some members, all members in a repetitive member system rarely experience such splits. Therefore, a C_H of greater than 1.0 should typically apply when repetitive framing or built-up members are used. For members with no splits C_H equals 2.0.

In addition, future allowable horizontal shear values will be increased by a factor of 2 or more because of a recent change in the applicable standard regarding assignment of strength properties. The change is a result of removing a conservative adjustment to the test data whereby a 50 percent reduction for checks and splits was applied in addition to a 4/9 stress concentration factor as described in Section 5.2.3. As an interim solution, a shear adjustment factor, C_H, of 2.0 should therefore apply to all designs that use horizontal shear values in 1997 and earlier editions of the NDS. As shown in Table 5.2, the C_H factor applies only to the allowable horizontal shear stress, F_v. As an interim consideration regarding horizontal shear at notches and connections in members, a C_H value of 1.5 is recommended for use with provisions in NDS•3.4.4 and 3.4.5 for dimension lumber only.

5.2.4.4 Size Factor (C_F)

Tabulated design values in the NDS-S are based on testing conducted on members of certain sizes. The specified depth for dimension lumber members subjected to testing is 12 inches for No. 3 or better, 6 inches for stud-grade members, and 4 inches for construction-, standard- or utility-grade members (i.e., C_F=1.0).

The size of a member affects unit strength because of the member's relationship to the likelihood of naturally occurring defects in the material.

Therefore, an adjustment to certain tabulated values is appropriate for sizes other than those tested; however, the tabulated values for Southern Yellow Pine have already been adjusted for size and do not require application of C_F. Table 5.2 indicates the tabulated values that should be adjusted to account for size differences. The adjustment applies when visually graded lumber is 2 to 4 inches thick or when a minimum 5-inch-thick rectangular bending member exceeds 12 inches in depth. Refer to NDS-S for the appropriate size adjustment factor.

5.2.4.5 Column Stability Factor (C_P)

Tabulated compression design values in the NDS-S are based on the assumption that a compression member is continuously supported along its length to prevent lateral displacement in both the weak and strong axes. When a compression member is subject to continuous lateral support in at least two orthogonal directions, Euler buckling cannot occur. However, many compression members (e.g., interior columns or wall framing) do not have continuous lateral support in two directions.

The column stability factor, C_P adjusts the tabulated compression stresses to account for the possibility of column buckling. For rectangular or non-symmetric columns, C_p must be determined for both the weak- and strong-axis bracing conditions. C_p is based on end-fixity, effective length of the member between lateral braces, and the cross-sectional dimensions of the member that affect the slenderness ratio used in calculating the critical buckling stress. Given that the Euler buckling effect is associated only with axial loads, the C_P factor applies to the allowable compressive stress parallel to grain, F_c, as shown in Table 5.2. Refer to the NDS for the equations used to calculate the column stability factor.

5.2.4.6 Beam Stability Factor (C_L)

The tabulated bending design values, F_b, given in the NDS-S are applicable to bending members that are either braced against lateral-torsional buckling (i.e., twisting) or stable without bracing (i.e., depth is no greater than the breadth of the member). Most bending members in residential construction are laterally supported on the compression edge by some type of sheathing product. The beam stability factor does, however, apply to conditions such as ceiling joists supporting unfinished attic space. When a member does not meet the lateral support requirements of NDS•3.3.3 or the stability requirements of NDS•4.4.1, the designer should modify the tabulated bending design values by using the beam stability factor, C_L, to account for the possibility of lateral-torsional buckling. For glued laminated timber bending members, the volume factor (C_V) and beam stability factor (C_L) are not applied simultaneously; thus, the lesser of these factors applies. Refer to the NDS•3.3.3 for the equations used to calculate C_L.

5.3 Structural Evaluation

As with any structural design, the designer should perform several checks with respect to various design factors. This section provides an overview of checks specified in the NDS and specifies several design concerns that are not addressed by the NDS. In general, the two categories of structural design concerns are:

Structural Safety (strength)

- Bending and lateral stability
- Horizontal Shear
- Bearing
- Combined bending and axial loading
- Compression and column stability
- Tension

Structural Serviceability

- Deflection due to bending
- Floor vibration
- Shrinkage

The remainder of this chapter applies these design checks to examples of different structural systems and elements in a home. In addition, given that the intent of this guide is to provide supplemental instruction for the use of the NDS in the efficient design of wood-framed homes, the reader is referred to the NDS for symbol definitions, as well as other guidance.

5.3.1 Structural Safety Checks

Bending (Flexural) Capacity

The following equations from the NDS determine if a member has sufficient bending strength. Notches in bending members should be avoided, but small notches are permissible; refer to NDS•3.2.3. Similarly, the diameter of holes in bending members should not exceed one-third the member's depth and should be located along the center line of the member. Greater flexural capacity may be obtained by increasing member depth, decreasing the clear span or spacing of the member, or selecting a grade and species of lumber with a higher allowable bending stress. Engineered wood products or alternative materials may also be considered.

[NDS•3.3]

$f_b \le F_b'$ basic design check for bending stress

$F_b' = F_b$ x (applicable adjustment factors per Section 5.2.4)

$f_b = \frac{Mc}{I} = \frac{M}{S}$ extreme fiber bending stress due to bending moment from transverse load

$S = \frac{I}{c} = \frac{bd^2}{6}$ section modulus of rectangular member

$I = \frac{bd^3}{12}$ moment of inertia of rectangular member

$c = \frac{1}{2}d$ distance from extreme fiber to neutral axis

Horizontal Shear

Because shear parallel to grain (i.e., horizontal shear) is induced by bending action, it is also known as bending shear and is greatest at the neutral axis. Bending shear is not transverse shear; lumber will always fail in other modes before failing in transverse or cross-grain shear owing to the longitudinal orientation of the wood fibers in structural members.

The horizontal shear force is calculated for solid sawn lumber by including the component of all loads (uniform and concentrated) that act perpendicular to the bearing surface of the solid member in accordance with NDS•3.4.3. Loads within a distance, *d*, from the bearing point are not included in the horizontal shear calculation; *d* is the depth of the member for solid rectangular members. Transverse shear is not a required design check, although it is used to determine the magnitude of horizontal shear by using basic concepts of engineering mechanics as discussed below.

The following equations from NDS•3.4 for horizontal shear analysis are limited to solid flexural members such as solid sawn lumber, glulam, or mechanically laminated beams. Notches in beams can reduce shear capacity and should be considered in accordance with NDS•3.4.4. Also, bolted connections influence the shear capacity of a beam; refer to NDS•3.4.5. If required, greater horizontal shear capacity may be obtained by increasing member depth or width, decreasing the clear span or spacing of the member, or selecting another species with a higher allowable shear capacity. The general equation for horizontal shear stress is discussed in the NDS and in mechanics of materials text books. Because dimension lumber is solid and rectangular, the simple equation for f_v is most commonly used.

[NDS•3.4]

$f_v \le F'_v$ basic design check for horizontal shear

$F'_v = F_v$ x (applicable adjustment factors per Section 5.2.4)

$f_v = \frac{VQ}{Ib}$ horizontal shear stress (general equation)

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

Compression Perpendicular to Grain (Bearing)

For bending members bearing on wood or metal, a minimum bearing of 1.5 inches is typically recommended. For bending members bearing on masonry, a minimum bearing of 3 inches is typically advised. The resulting bearing areas may not, however, be adequate in the case of heavily loaded members. On the other hand, they may be too conservative in the case of lightly loaded members. The minimum bearing lengths are considered to represent good practice.

The following equations from the NDS are based on net bearing area. Note that the provisions of the NDS acknowledge that the inner bearing edge experiences added pressure as the member bends. As a practical matter, the added pressure does not pose a problem because the compressive capacity, $F'_{c\perp}$, of wood increases as the material is compressed. Further, the design value is based on a deformation limit, not on failure by crushing. Thus, the NDS recommends the added pressure at bearing edges not be considered. The designer is also alerted to the use of the bearing area factor, C_b, which accounts for the ability of wood to distribute large stresses originating from a small bearing area not located near the end of a member. Examples include interior bearing supports and compressive loads on washers in bolted connections.

[NDS•3.10]

$f_{c\perp} \leq F'_{c\perp}$ basic design check for compression perpendicular to grain

$F'_{c\perp} = F_{c\perp}$ x (applicable adjustment factors per Section 5.2.4)

$f_{c\perp} = \dfrac{P}{A_b}$ stress perpendicular to grain due to load, P, on net bearing area, A_b.

The above equations pertain to bearing that is perpendicular to grain; for bearing at an angle to grain, refer to NDS•3.10. The later condition would apply to sloped bending members (i.e., rafters) notched at an angle for bearing. For light-frame construction, bearing stress is rarely a limiting factor.

Combined Bending and Axial Loading

Depending on the application and the combination of loads considered, some members such as wall studs and roof truss members, experience bending stress in addition to axial loading. The designer should evaluate combined bending and axial stresses as appropriate. If additional capacity is required, the selection of a higher grade of lumber is not always an efficient solution for overstressed compression members under combined axial and bending loads because the design may be limited by stability rather than by a stress failure mode. Efficiency issues will become evident when the designer calculates the components of the combined stress interaction equations that are given below and found in the NDS.

[NDS•3.9]

Combined bending and axial tension design check

$$\frac{f_t}{F'_t} + \frac{f_b}{F^*_b} \leq 1$$

$$\frac{f_b - f_t}{F^{**}_b} \leq 1$$

Combined bending and axial compression design check

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b1}}{F'_{b1}\left(1 - {}^{f_c}\!/\!_{F_{cE1}}\right)} + \frac{f_{b2}}{F'_{b2}\left(1 - \left({}^{f_c}\!/\!_{F_{cE2}}\right) - \left({}^{f_{b1}}\!/\!_{F_{bE}}\right)^2\right)} \leq 1$$

Compression and Column Stability

For framing members that support axial loads only (i.e., columns), the designer must consider whether the framing member can withstand the axial compressive forces on it without buckling or compressive failure. If additional compression strength is required, the designer should increase member size, decrease framing member spacing, provide additional lateral support, or select a different grade and species of lumber with higher allowable stresses. Improving lateral support is usually the most efficient solution when stability controls the design (disregarding any architectural limitations). The need for improved lateral support will become evident when the designer performs the calculations necessary to determine the stability factor, C_P, in accordance with NDS•3.7. When a column has continuous lateral support in two directions, buckling is not an issue and C_p = 1.0. If, however, the column is free to buckle in one or more directions, C_p must be evaluated for each direction of possible buckling. The evaluation must also consider the spacing of intermediate bracing, if any, in each direction.

[NDS•3.7]

$f_c \le F'_c$ basic design check for compression parallel to grain

$F'_c = F_c$ x (applicable adjustment factors from Section 5.2.4, including C_p)

$f_c = \dfrac{P}{A}$ compressive stress parallel to grain due to axial load, P, acting on the member's cross-sectional area, A.

$$C_p = \frac{1+\left(F_{cE}/F_c^{*}\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^{*}\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^{*}}{c}}$$ column stability factor

$$F_{cE} = \frac{K_{cE}E'}{\left(\ell_e/d\right)^2}$$

$F_c^{*} = F_c$ x (same adjustment factors for F'_c except C_p is not used)

Tension

Relatively few members in light-frame construction resist tension forces only. One notable exception occurs in roof framing where cross-ties or bottom chords in trusses primarily resist tension forces. Other examples include chord and collector members in shear walls and horizontal diaphragms as discussed in Chapter 6. Another possibility is a member subject to excessive uplift loads such as those produced by extreme wind. In any event, connection design is usually the limiting factor in designing the transfer of tension forces in light-frame construction (refer to Chapter 7). Tension stresses in wood members are checked by using the equations below in accordance with NDS•3.8.

[NDS•3.8]

$f_t \le F'_t$ basic design check for tension parallel to grain

$F'_t = F_t$ x (applicable adjustment factors per Section 5.2.4)

$f_t = \dfrac{P}{A}$ stress in tension parallel to gain due to axial tension load, P, acting on the member's cross-sectional area, A

The NDS does not provide explicit methods for evaluating cross-grain tension forces and generally recommends the avoidance of cross-grain tension in lumber even though the material is capable of resisting limited cross-grain stresses. Design values for cross-grain tension may be approximated by using one-third of the unadjusted horizontal shear stress value, F_v. One application of cross-grain tension in design is in the transfer of moderate uplift loads from wind through the band or rim joist of a floor to the construction below. If additional cross-grain tension strength is required, the designer should increase member size or consider alternative construction details that reduce cross-grain tension forces. When excessive tension stress perpendicular to grain cannot be avoided, the use of mechanical reinforcement or design detailing to reduce the cross-grain tension forces is considered good practice (particularly in high-hazard seismic regions) to ensure that brittle failures do not occur.

5.3.2 Structural Serviceability

Deflection Due to Bending

The NDS does not specifically limit deflection but rather defers to designer judgment or building code specifications. Nonetheless, with many interior and exterior finishes susceptible to damage by large deflections, reasonable deflection limits based on design loads are recommended herein for the design of specific elements.

The calculation of member deflection is based on the section properties of the beam from NDS-S and the member's modulus of elasticity with applicable adjustments. Generally, a deflection check using the equations below is based on the estimated maximum deflection under a specified loading condition. Given that wood exhibits time- and load-magnitude-dependent permanent deflection (creep), the total long-term deflection can be estimated in terms of two components of the load related to short- and long-term deflection using recommendations provided in NDS•3.5.

[NDS•3.5]

$$\Delta_{estimate} \le \Delta_{allow} = \frac{\ell}{(120 \text{ to } 600)} \quad \text{(see Table 5.5 for value of denominator)}$$

$$\Delta_{estimate} \cong f\left(\frac{\text{load and span}}{EI}\right) \quad \text{(see beam equations in Appendix A)}$$

If a deflection check proves unacceptable, the designer may increase member depth, decrease the clear span or spacing of the member, or select a grade and species of wood with a higher modulus of elasticity (the least effective option). Typical denominator values used in the deflection equation range from 120 to 600 depending on application and designer judgment. Table 5.5 provides recommended deflection limits. Certainly, if a modest adjustment to a deflection limit results in a more efficient design, the designer should exercise discretion with respect to a possible negative consequence such as vibration or long-term creep. For lateral bending loads on walls, a serviceability load for a deflection check may be considered as a fraction of the nominal design wind load for exterior walls. A reasonable serviceability wind load criteria may be taken as 0.75W or 75 percent of the nominal design wind load (Galambos and Ellingwood, 1986).

TABLE 5.5 Recommended Allowable Deflection Limits[1]

Element or Condition	Deflection Limit, Δ_{all} [2]	Load Condition
Rafters without attached ceiling finish	$\ell/180$	L_r or S
Rafters with attached ceiling finishes and trusses	$\ell/240$	L_r or S
Ceiling joists with attached finishes	$\ell/240$	L_{attic}
Roof girders and beams	$\ell/240$	L_r or S
Walls	$\ell/180$	W or E
Headers	$\ell/240$	(L_r or S) or L
Floors[3]	$\ell/360$	L
Floor girders and beams[4]	$\ell/360$	L

Notes:
[1]Values may be adjusted according to designer discretion with respect to potential increases or decreases in serviceability. In some cases, a modification may require local approval of a code variance. Some deflection checks may be different or not required depending on the local code requirements. The load condition includes the live or transient load only, not dead load.
[2]ℓ is the clear span in units of inches for deflection calculations.
[3]Floor vibration may be controlled by using $\ell/360$ for spans up to 15 feet and a 1/2-inch limit for spans greater than 15 feet. Wood I-joist manufacturers typically recommend $\ell/480$ as a deflection limit to provide enhanced floor performance and to control nuisance vibrations.
[4]Floor vibration may be controlled for combined girder and joist spans of greater than 20 feet by use of a $\ell/480$ to $\ell/600$ deflection limit for the girder.

Given that system effects influence the stiffness of assemblies in a manner similar to that of bending capacity (see Section 5.2.4.2), the system deflection factors of Table 5.6 are recommended. The estimated deflection based on an analysis of an element (e.g., stud or joist) is multiplied by the deflection factors to account for system effect. Typical deflection checks on floors under uniform loading can be easily overestimated by 20 percent or more. In areas where partitions add to the rigidity of the supporting floor, deflection can be overestimated by more than 50 percent (Hurst, 1965). When concentrated loads are considered on typical light-frame floors with wood structural panel subflooring, deflections can be overestimated by a factor of 2.5 to 3 due to the neglect of the load distribution to adjacent framing members and partial composite action (Tucker and Fridley, 1999). Similar results have been found for sheathed wall assemblies (NAHBRF, 1974). When adhesives attach wood structural panels to wood framing, even greater reductions in deflection are realized due to increased composite action (Gillespie et al., 1978; Pellicane and Anthony, 1996). However, if a simple deflection limit such as $\ell/360$ is construed to control floor vibration in addition to the serviceability of finishes, the use of system deflection factors of Table 5.6 is not recommended for floor system design. In this case, a more accurate estimate of actual deflection may result in a floor with increased tendency to vibrate or bounce.

TABLE 5.6 System Deflection Adjustment Factors[1]

Framing System	Multiply single member deflection estimate by:
Light-wood-frame floor system with minimum 2x8 joists, minimum 3/4-inch-thick sheathing,[2] and standard fastening	0.85–Uniform load 0.4–Concentrated load
Light-wood-frame floor system as above, but with glued and nailed sheathing	0.75–Uniform load 0.35–Concentrated load
Light-wood-frame wall system with 2x4 or 2x6 studs with minimum 3/8-inch-thick sheathing on one side and 1 /2-inch-thick gypsum board on the other; both facings applied with standard fastening[3]	0.7–2x4 0.8–2x6

Notes:
[1]System deflection factors are not recommended when evaluating floor member deflection limits of Table 5.5 with the implied purpose of controlling floor vibration.
[2]Two sheathing layers may be used to make up a minimum thickness of 3/4-inch.
[3]The factors may be adjusted according to fastener diameter in accordance with footnote 5 of Table 5.4. If fastening is doubled (i.e., spacing halved), the factors may be divided by 1.4 (Polensek, 1975).

Floor Vibration

The NDS does not specifically address floor vibration because it is a serviceability rather than a safety issue. In addition, what is considered an “acceptable” amount of floor vibration is highly subjective. Accordingly, reliable design information on controlling floor vibration to meet a specific level of “acceptance” is not readily available; therefore, some rules of thumb are provided below for the designer wishing to limit vibration beyond that implied by the traditional use of an $\ell/360$ deflection limit (FHA, 1958; Woeste and Dolan, 1998).

- For floor joist spans less than 15 feet, a deflection limit of $\ell/360$ considering design live loads only may be used, where ℓ is the clear span of the joist in inches.
- For floor joist clear spans greater than 15 feet, the maximum deflection should be limited to 0.5 inches.
- For wood I-joists, the manufacturer’s tables that limit deflection to $\ell/480$ should be used for spans greater than 15 feet, where ℓ is the clear span of the member in inches.
- When calculating deflection based on the above rules of thumb, the designer should use a 40 psf live load for all rooms whether or not they are considered sleeping rooms.
- As an additional recommendation, glue and mechanically fasten the floor sheathing to the floor joists to enhance the floor system’s strength and stiffness.

Floor deflections are typically limited to $\ell/360$ in the span tables published in current building codes using a standard deflection check without consideration of system effects. For clear spans greater than 15 feet, this deflection limit has caused nuisance vibrations that are unacceptable to some building occupants or owners. Floor vibration is also aggravated when the floor is supported on a bending member (e.g., girder) rather than on a rigid bearing wall. It may be desirable to design such girders with a smaller deflection limit to control floor vibration, particularly when girder and floor spans have more than a 20-foot total combined span (i.e., span of girder plus span of supported floor joist).

For metal-plate-connected wood trusses, strong-backs are effective in reducing floor vibration when they are installed through the trusses near the center of the span. A strong-back is a continuous bracing member, typically a 2x6, fastened edgewise to the base of the vertical web of each truss with 2-16d nails. For longer spans, strong-backs may be spaced at approximately 8-foot intervals across the span. Details for strong-backs may be found in the *Metal Plate Connected Wood Truss Handbook* (WTCA, 1997). Alternatively, a more stringent deflection criteria may be used for the floor truss design.

Shrinkage

The amount of wood shrinkage in a structure depends on the moisture content (MC) of the lumber at the time of installation relative to the equilibrium moisture content (EMC) that the wood will ultimately attain in use. It is also dependent on the detailing of the structure such as the amount of lumber supporting loads in a perpendicular-to-grain orientation (i.e., sill, sole, top plates, and joists). MC at installation is a function of the specified drying method, jobsite storage practices, and climate conditions during construction. Relatively dry lumber (15 percent or less) minimizes shrinkage problems affecting finish materials and prevents loosening or stressing of connections. A less favorable but acceptable alternative is to detail the structure such that shrinkage is uniform, dispersed, or otherwise designed to minimize problems. This alternative is the "defacto" choice in simple residential buildings.

Shrink and swell across the width or thickness of lumber can be estimated by the equation below from ASTM D1990 for typical softwood structural lumber (ASTM, 1998a). Shrinkage in the longitudinal direction of the member is practically negligible.

[ASTM D1990•App. X.1]

$$d_2 = d_1\left(\frac{1-\frac{a-0.2M_2}{100}}{1-\frac{a-0.2M_1}{100}}\right)$$

d_1 = member width or thickness at moisture content M_1

d_2 = member width or thickness at moisture content M_2

$a = 6.0$ (for width dimension)

$a = 5.1$ (for thickness dimension)

5.4 Floor Framing

The objectives of floor system design are

- to support occupancy live loads and building dead loads adequately;
- to resist lateral forces resulting from wind and seismic loads and to transmit the forces to supporting shear walls through diaphragm action;
- to provide a suitable subsurface for floor finishes;
- to avoid owner complaints (e.g., excessive vibration, noise, etc.);
- to serve as a thermal barrier over unconditioned areas (e.g., crawl spaces); and
- to provide a one- to two-hour fire rating between dwelling units in multifamily buildings (refer to local building codes).

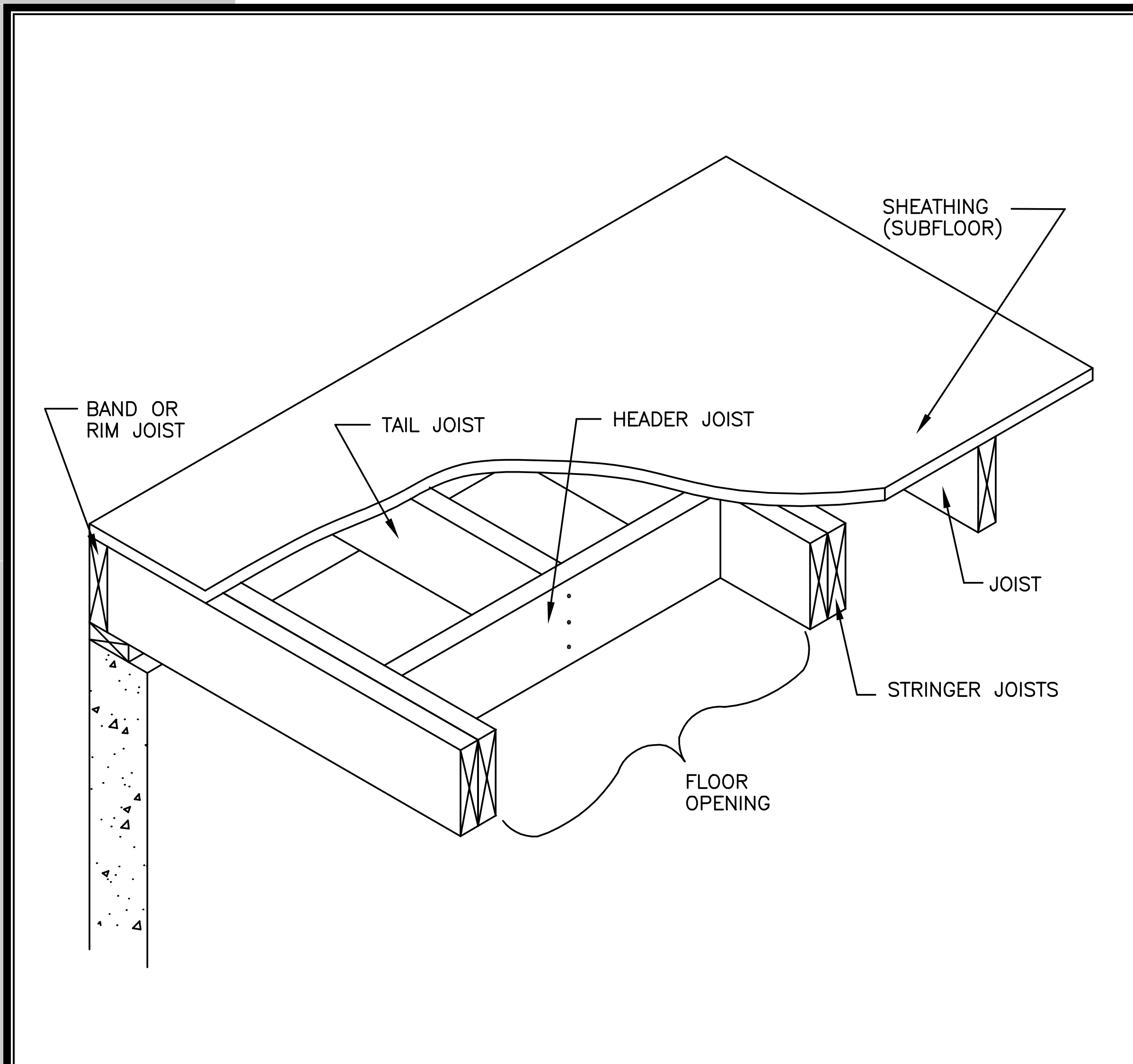
5.4.1 General

A wood floor is a horizontal structural system composed primarily of the following members:

- joists;
- girders; and
- sheathing.

Wood floor systems have traditionally been built of solid sawn lumber for floor joists and girders, although parallel chord wood trusses and wood I-joists are seeing increasing use, and offer advantages for dimensional consistency, and spans. Floor joists are horizontal, repetitive framing members that support the floor sheathing and transfer the live and dead floor loads to the walls, girders, or columns below. Girders are horizontal members that support floor joists not otherwise supported by interior or exterior load-bearing walls. Floor sheathing is a horizontal structural element, usually plywood or oriented strand board panels, that directly supports floor loads and distributes the loads to the framing system below. Floor sheathing also provides lateral support to the floor joists. As a structural system, the floor provides resistance to lateral building loads resulting from wind and seismic forces and thus constitutes a “horizontal diaphragm” (refer to Chapter 6). Refer to Figure 5.2 for an illustration of floor system structural elements and to *Cost-Effective Home Building: A Design and Construction Handbook* for efficient design ideas and concepts (NAHBRC, 1994).

FIGURE 5.2 ***Structural Elements of the Floor System***



The design approach discussed herein addresses solid sawn lumber floor systems in accordance with the procedures specified in the *National Design Specification for Wood Construction* (NDS), with appropriate modifications as noted. For more information regarding wood I-joists, trusses, and other materials, consult the manufacturer's specifications and applicable code evaluation reports.

Section 5.3 discusses the general design equations and design checks for the NDS. The present section provides detailed design examples that apply the equations in Section 5.3, while tailoring them to the design of the elements in a floor system. The next sections make reference to the span of a member. The NDS defines span as the clear span of the member plus one-half the required bearing at each end of the member. This guide simply defines span as the clear span between bearing points.

When designing any structural element, the designer must first determine the loads acting on the element. Load combinations used in the analysis of floor members in this guide are taken from Table 3.1 of Chapter 3. Given that only the dead loads of the floor system and live loads of occupancy are present in a typical floor system, the controlling design load combination for a simply-supported floor joist is D+L. For joists with more complicated loading, such as cantilevered joists supporting roof framing, the following load combinations may be considered in accordance with Chapter 3:

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

5.4.2 Floor Joist Design

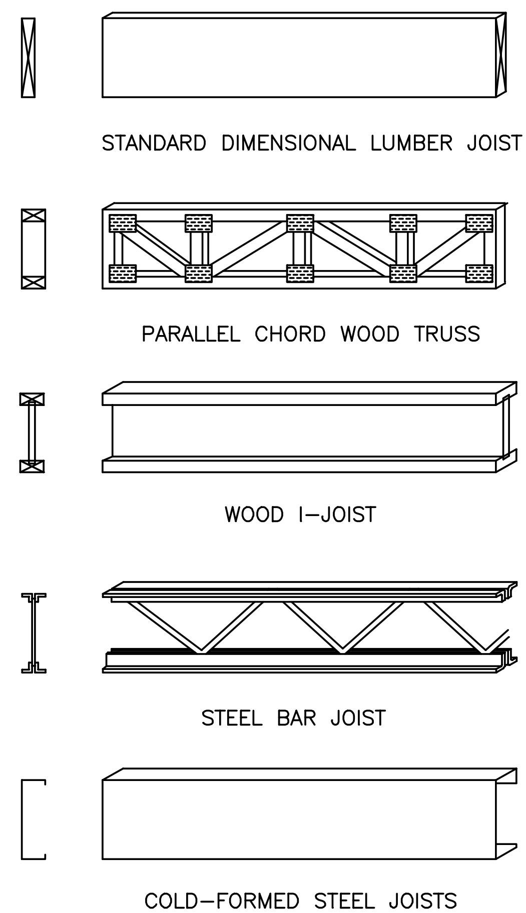
Readily available tables in residential building codes provide maximum allowable spans for different species, grades, sizes, and spacings of lumber joists. Some efficient concepts for floor joist design are also provided in *Cost Effective Home Building: A Design and Construction Handbook* (NAHB, 1994). Therefore, it is usually not necessary to design conventional floor joists for residential construction. To obtain greater economy or performance, however, designers may wish to create their own span tables or spreadsheets for future use in accordance with the methods shown in this section.

Keep in mind that the grade and species of lumber is often a regional choice governed by economics and availability; some of the most common species of lumber for floor joists are Hem-Fir, Spruce-Pine-Fir, Douglas-Fir, and Southern Yellow Pine. Bear in mind, too, that the most common sizes for floor joists are 2x8 and 2x10, although 2x12s are also frequently used. The following examples are located in Section 5.7 and illustrate the design of typical floor joists in accordance with the principles discussed earlier:

- simple span joist (Examples 5.1 and 5.2); and
- cantilevered joist (Example 5.3).

For different joist applications, such as a continuous multiple span, the designer should use the appropriate beam equations (refer to Appendix A) to estimate the stresses induced by the loads and reactions. Other materials such as wood I-joists and parallel chord floor trusses are also commonly used in light-frame residential and commercial construction; refer to the manufacturer's data for span tables for wood I-joists and other engineered wood products. For additional information on wood floor trusses that can be ordered to specification with engineering certification (i.e., stamped shop drawings), refer to Section 5.6.3 on roof trusses. Cold-formed steel floor joists or trusses may also be considered. Figure 5.3 illustrates some conventional and alternative floor joist members.

FIGURE 5.3 Conventional and Alternative Floor Framing Members



Notes:
[1]Trusses are also available with trimmable ends.
[2]Cold-formed steel is also used to make floor trusses.

For typical floor systems supporting a concentrated load at or near center span, load distribution to adjacent joists can substantially reduce the bending stresses or moment experienced by the loaded joist. A currently available design methodology may be beneficial for certain applications such as wood-framed garage floors that support heavy concentrated wheel loads (Tucker and Fridley, 1999). Under such conditions, the maximum bending moment experienced by any single joist is reduced by more than 60 percent. A similar reduction in the shear loading (and end reaction) of the loaded joist also results, with exception for "moving" concentrated loads that may be located near the end of the joist, thus creating a large transverse shear load with a small bending moment. The above-mentioned design methodology for a single, concentrated load applied near mid-span of a repetitive member floor system is essentially equivalent to using a C_r factor of 1.5 or more (see Section 5.2.4.2). The system deflection adjustment factors in Table 5.6 are applicable as indicated for concentrated loads.

Bridging or cross-braces were formerly thought to provide both necessary lateral-torsional bracing of dimension lumber floor joists and stiffer floor systems.

However, full-scale testing of 10 different floor systems as well as additional testing in completed homes has conclusively demonstrated that bridging or cross-bracing provides negligible benefit to either the load-carrying capacity or stiffness of typical residential floors with dimension lumber framing (sizes of 2x6 through 2x12) and wood structural panel subflooring (NAHB, 1961). These same findings are not proven to apply to other types of floor joists (i.e., I-joists, steel joists, etc.) or for dimension lumber joists greater than 12 inches in depth. According to the study, bridging may be considered necessary for 2x10 and 2x12 dimension lumber joists with clear spans exceeding about 16 feet and 18 feet, respectively (based on a 50 psf total design load and L/360 deflection limit). To the contrary, the beam stability provisions of NDS•4.4.1 conservatively require bridging to be spaced at intervals not exceeding 8 feet along the span of 2x10 and 2x12 joists.

5.4.3 Girder Design

The decision to use one girder over another is a function of cost, availability, span and loading conditions, clearance or head-room requirements, and ease of construction. Refer to the Figure 5.4 for illustrations of girder types. Girders in residential construction are usually one of the following types:

- built-up dimension lumber;
- steel I-beam;
- engineered wood beam;
- site-fabricated beam;
- wood I-joist; or
- metal plate connected wood truss.

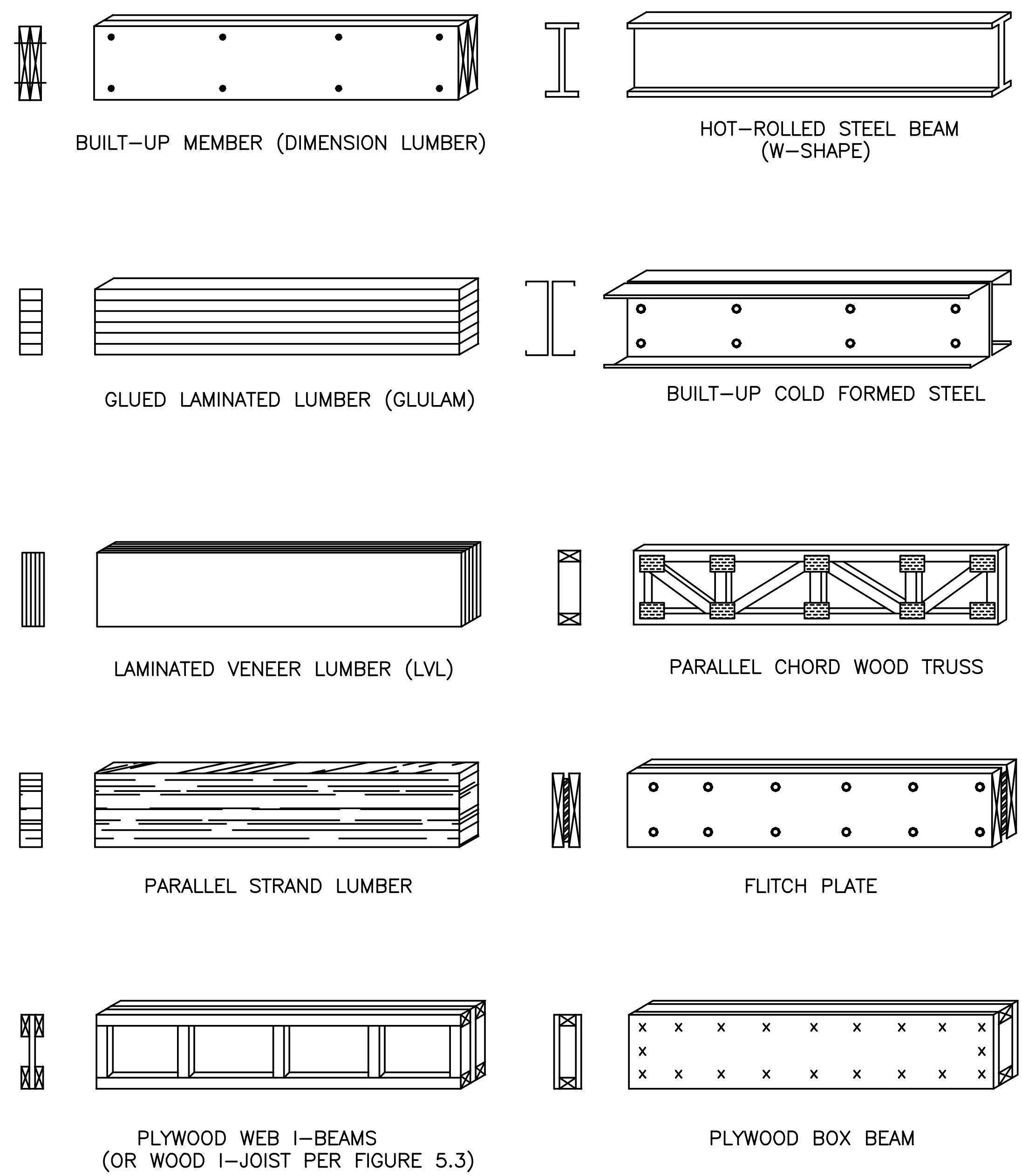
Built-up beams are constructed by nailing together of two or more plys of dimension lumber. Since load sharing occurs between the plys (i.e., lumber members), the built-up girder is able to resist higher loads than a single member of the same overall dimensions. The built-up member can resist higher loads only if butt joints are located at or near supports and are staggered in alternate plys. Each ply may be face nailed to the previous ply with 10d nails staggered at 12 inches on center top to bottom. The design method and equations are the same as those in Section 5.4.2 for floor joists; however, the adjustment factors applying to design values and loading conditions are somewhat different. The designer needs to keep the following in mind:

- Although floor girders are not typically thought of as “repetitive” members, a repetitive member factor is applicable if the floor girder is built-up from two or more members (three or more according to the NDS).

- The beam stability factor, C_L, is determined in accordance with NDS•3.3.3; however, for girders supporting floor framing, lateral support is considered to be continuous and $C_L = 1$.

Example 5.4 illustrates the design of a built-up floor girder.

FIGURE 5.4 ***Examples of Beams and Girders***



Steel I beams are often used in residential construction because of their greater spanning capability. Compared with wood members, they span longer distances with a shallower depth. A 2x4 or 2x6 is usually attached to the top surface with bolts to provide a fastening surface for floor joists and other structural members. Although steel beam shapes are commonly referred to as I-beams, a typical 8-inch-deep W-shaped beam is commonly considered a house beam. Alternatively, built-up cold-formed steel beams (i.e., back-to-back C-shapes) may be used to construct I-shaped girders. Refer to the *Steel Construction Manual* (AISC, 1989) and the American Iron and Steel Institute's publication RG-936 for the design of and span tables for residential applications of hot-rolled steel sections (AISI, 1993). Structural steel floor beam span tables are also found in the *Beam Series* (NAHBRC, 1981). The *Prescriptive Method for Cold-Formed Steel in Residential Construction* should be consulted for the design of built-up cold-formed steel sections as headers and girders (NAHBRC, 1998).

Engineered wood beams include I-joists, wood trusses (i.e., girder trusses) glue-laminated lumber, laminated veneer lumber, parallel strand lumber, etc. This guide does not address the design of engineered wood girders because product manufacturers typically provide span tables or engineered designs that are considered proprietary. Consult the manufacturer for design guidelines or completed span tables. The NDS does, however, provide a methodology for the design of glue-laminated beams (NDS•5).

Site-fabricated beams include plywood box beams, plywood I-beams, and flitch plate beams. *Plywood box beams* are fabricated from continuous dimension lumber flanges (typically 2x4s or 2x6s) sandwiched between two plywood webs; stiffeners are placed at concentrated loads, end bearing points, plywood joints, and maximum 24-inch intervals. *Plywood I-beams* are similar to box beams except that the plywood web is sandwiched between dimension lumber wood flanges (typically 2x4s or 2x6s), and stiffeners are placed at maximum 24-inch intervals. *Flitch plate beams* are fabricated from a steel plate sandwiched between two pieces of dimension lumber to form a composite section. Thus, a thinner member is possible in comparison to a built-up wood girder of similar strength. The steel plate is typically 1/4 to 1/2 inches thick and about 1/4-inch less in depth than the dimension lumber. The sandwich construction is usually assembled with through-bolts staggered at about 12 inches on center. Flitch plate beams derive their strength and stiffness from the composite section of steel plate and dimension lumber. The lumber also provides a medium for fastening other materials using nails or screws.

Span tables for plywood I-beams, plywood box beams, steel-wood I-beams, and flitch plate beams are provided in NAHB's *Beam Series* publications (NAHBRC, 1981). Refer to the APA's *Product Design Specification* (PDS) and *Supplement* for the design method used for plywood box beams (APA, 1998b). The *International One- and Two-Family Dwelling Code* (ICC, 1998), formerly the *CABO One- and Two-Family Dwelling Code*, provides a simple prescriptive table for plywood box beam headers.

5.4.4 Subfloor Design

Typical subfloor sheathing is nominal 5/8- or 3/4-inch-thick 4x8 panels of plywood or oriented strand board (OSB) with tongue-and-groove edges at unsupported joints perpendicular to the floor framing. Sheathing products are generally categorized as wood structural panels and are specified in accordance with the prescriptive span rating tables published in a building code or are made available by the manufacturer. Example 5.5 uses the *Design and Construction Guide: Residential and Commercial* (APA, 1998a) to specify sheathing. The prescriptive tables provide maximum spans (joist spacing) based on sheathing thickness and span rating. It is important to note that the basis for the prescriptive tables is the standard beam calculation. If loads exceed the limits of the prescriptive tables, the designer may be required to perform calculations; however, such calculations are rarely necessary. In addition, the APA offers a plywood floor guide for residential garages that assists in specifying plywood subflooring suitable for heavy concentrated loads from vehicle tire loading (APA, 1980).

The APA also recommends a fastener schedule for connecting sheathing to floor joists. Generally, nails are placed a minimum of 6 inches on center at edges and 12 inches on center along intermediate supports. Refer to Table 5.7 for recommended nail sizes based on sheathing thickness. Nail sizes vary with nail type (e.g., sinkers, box nails, and common nails), and various nail types have different characteristics that affect structural properties (refer to Chapter 7). For information on other types of fasteners, consult the manufacturer. In some cases, shear loads in the floor diaphragm resulting from lateral loads (i.e., wind and earthquake) may require a more stringent fastening schedule; refer to Chapter 6 for a discussion on fastening schedules for lateral load design. Regardless of fastener type, gluing the floor sheathing to the joists increases floor stiffness and strength.

TABLE 5.7 Fastening Floor Sheathing to Structural Members[1]

Thickness	Size and Type of Fastener
Plywood and wood structural panels, subfloor sheathing to framing	
1/2-inch and less	6d nail
19/32- to 1-inch	8d nail
1-1/8- to 1-1/4-inch	10d nail or 8d deformed shank nail
Plywood and wood structural panels, combination subfloor/underlayment to framing	
3/4-inch and less	8d nail or 6d deformed shank nail
7/8- to -inch	8d nail
1-1/8- to 1-1/4-inch	10d nail or 8d deformed shank nail

Notes:
[1]Codes generally require common or box nails; if pneumatic nails are used, as is common, refer to NER-272 (NES, 1997) or the nail manufacturer's data. Screws are also commonly substituted for nails. For more detail on fasteners and connections, refer to Chapter 7.

While not as common today, boards may also be used as a subfloor (i.e., board sheathing). Floor sheathing boards are typically 1x6 or 1x8 material laid flatwise and diagonally (or perpendicular) on the floor joists. They may be designed using the NDS or local accepted practice.

5.5 Wall Framing

The objectives of wall system design are

- to resist snow, live, and dead loads and wind and seismic forces;
- to provide an adequate subsurface for wall finishes and to provide openings for doors and windows;
- to serve as a thermal and weather barrier;
- to provide space and access for electrical and mechanical equipment, where required; and
- to provide a one- to two-hour fire barrier if the wall separates individual dwelling units in attached or multifamily buildings.

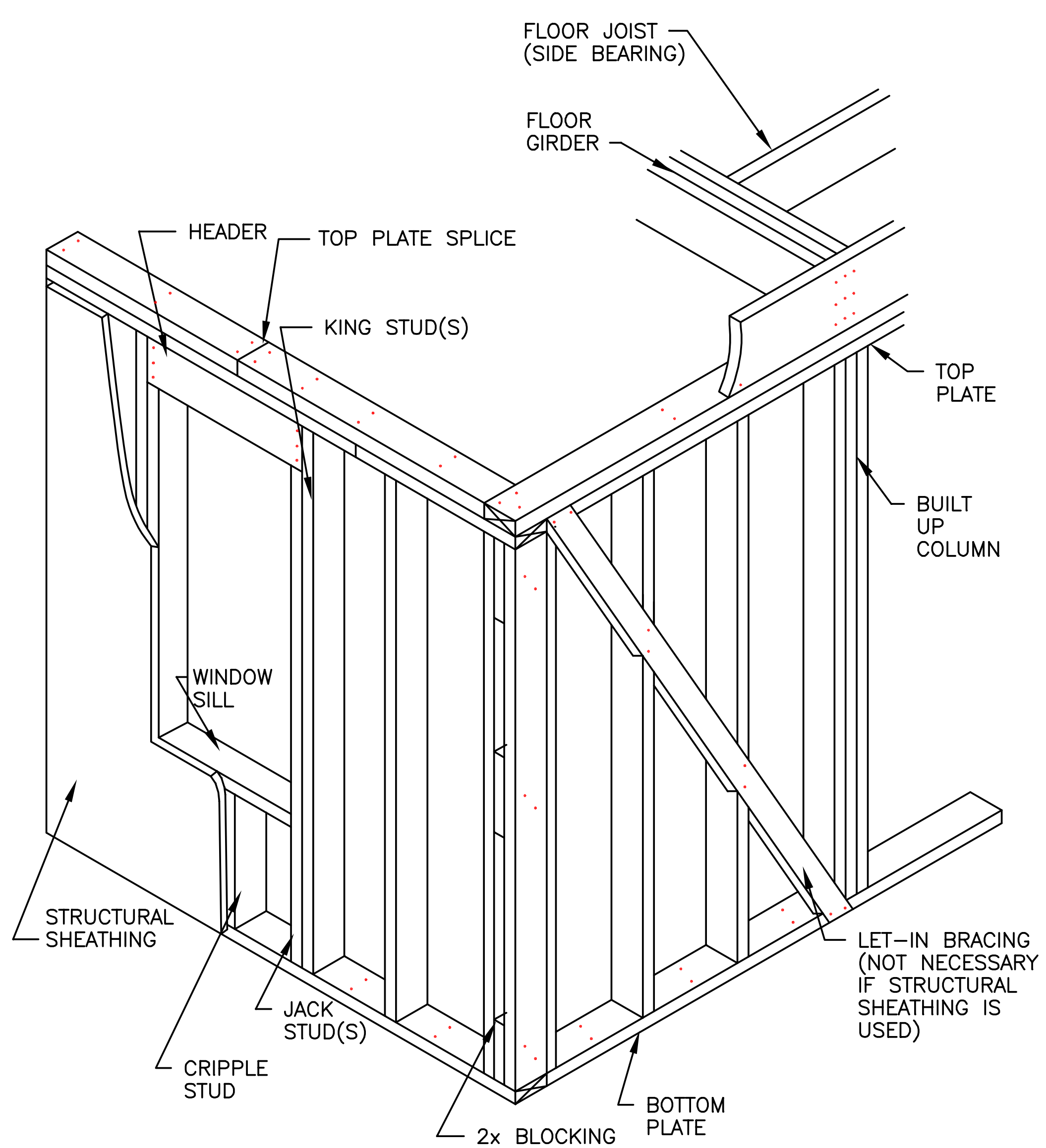
5.5.1 General

A wall is a vertical structural system that supports gravity loads from the roof and floors above and transfers the loads to the foundation below. It also resists lateral loads resulting from wind and earthquakes. A typical wood-framed wall is composed of the following elements as shown in Figure 5.5:

- studs, including wall, cripple, jack, and king studs;
- top and bottom (sole) plates;
- headers;
- sheathing; and
- diagonal let-in braces, if used.

Residential wall systems have traditionally been constructed of dimension lumber, usually 2x4s or 2x6s, although engineered wood studs and cold-formed steel studs are now seeing increased use. Wall studs are vertical, repetitive framing members spaced at regular intervals to support the wall sheathing. They span the full height of each story and support the building loads above. King and jack studs (also known as jamb studs) frame openings and support loads from a header. Cripple studs are placed above or below a wall opening and are not full height. Built-up wall studs that are assembled on the jobsite may be used within the wall to support concentrated loads. Top and bottom plates are horizontal members to which studs are fastened. The top and bottom plates are then fastened to the floor or roof above and either to the floor below or directly to the foundation. Headers are beams that transfer the loads above an opening to jack studs at each side of the opening.

FIGURE 5.5 Structural Elements of the Wall System



Structural wall sheathing, such as plywood or oriented strand board, distributes lateral loads to the wall framing and provides lateral support to both the wall studs (i.e., buckling resistance) and the entire building (i.e., racking resistance). Interior wall finishes also provide significant support to the wall studs and the structure. In low-wind and low-hazard seismic areas, metal ‘T’ braces or wood let-in braces may be used in place of wall sheathing to provide resistance to lateral (i.e., racking) loads. About 50 percent of new homes constructed each year now use wood structural panel braces, and many of those homes are fully-sheathed with wood structural panels. These bracing methods are substantially stronger than the let-in brace approach; refer to Chapter 6 for greater detail on the design of wall bracing. Wood let-in braces are typically 1x4 wood members that are “let-in” or notched into the studs and nailed diagonally across wall sections at corners and specified intervals. Their use is generally through application of conventional construction provisions found in most building codes for residential construction in combination with interior and exterior claddings.

The design procedure discussed herein addresses dimension lumber wall systems according to the *National Design Specification for Wood Construction* (NDS). Where appropriate, modifications to the NDS have been incorporated and are noted. Standard design equations and design checks for the NDS procedure were presented earlier in this chapter. The detailed design examples in this section illustrate the application of the equations by tailoring them to the design of the elements that make up residential wall systems.

Wall systems are designed to withstand dead and live gravity loads acting parallel to the wall stud length, as well as lateral loads–primarily wind and earthquake loads–acting perpendicular to the face of the wall. Wind also induces uplift loads on the roof; when the wind load is sufficient to offset dead loads, walls and internal connections must be designed to resist tension or uplift forces. The outcome of the design of wall elements depends on the degree to which the designer uses the “system strength” inherent in the construction. To the extent possible, guidance on system design in this section uses the NDS and the recommendations in Sections 5.2 and 5.3.

When designing wall elements, the designer needs to consider the load combinations discussed in Chapter 3, particularly the following ASD combinations of dead, live, snow, and wind loads:

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

A wall system may support a roof only or a roof and one or more stories above. The roof may or may not include an attic storage live load. A 10 psf attic live load used for the design of ceiling joists is intended primarily to provide safe access to the attic, not storage. The controlling load combination for a wall that supports only a roof is the second load combination listed above. If the attic is not intended for storage, the value for *L* should be 0. The controlling load combination for a wall that supports a floor, wall, and a roof should be either the first or second load combination depending on the relative magnitude of floor and roof snow loads.

The third load combination provides a check for the out-of-plane bending condition due to lateral wind loads on the wall. For tall wood-frame walls that support heavy claddings such as brick veneer, the designer should also consider out-of-plane bending loads resulting from an earthquake load combination, although the other load combinations above usually control the design. The third and fourth load combinations are essentially combined bending and axial loads that may govern stud design as opposed to axial load only in the first two load combinations. Chapter 6 addresses the design of walls for in-plane shear or racking forces resulting from lateral building loads caused by wind or earthquakes.

In many cases, certain design load combinations or load components can be dismissed or eliminated through practical consideration and inspection. They are a matter of designer judgment, experience, and knowledge of the critical design conditions.

5.5.2 Load-Bearing Walls

Exterior load-bearing walls support both axial and lateral loads. For interior load-bearing walls, only gravity loads are considered. A serviceability check using a lateral load of 5 psf is sometimes applied independently to interior walls but should not normally control the design of load-bearing framing. This section focuses on the axial and lateral load-bearing capacity of exterior and interior walls.

Exterior walls are not necessarily load-bearing walls. Load-bearing walls support gravity loads from either the roof, ceiling, or floor joists or the beams above. A gable-end wall is typically considered to be a nonload-bearing wall in that roof and floor framing generally runs parallel to the gable end; however, it must support lateral wind and seismic loads and even small dead and live loads. Exterior load-bearing walls must be designed for axial loads as well as for lateral loads from wind or seismic forces. They must also act as shear walls to resist racking loads from lateral wind or seismic forces on the overall building (refer to Chapter 6). Example 5.6 demonstrates the design of an exterior bearing wall.

When calculating the column stability factor for a stud wall, note that column capacity is determined by using the slenderness ratio about the strong axis of the stud $(l_e/d)_x$ in accordance with NDS•3.7.1. The reason for using the strong axis slenderness ratio is that lateral support is provided to the stud by the wall sheathing and finish materials in the stud's weak-axis bending or buckling direction. When determining the column stability factor, C_p, for a wall system rather than for a single column in accordance with NDS•3.7.1, the designer must exercise judgment with respect to the calculation of the effective length, ℓ_e, and the depth or thickness of the wall system, d. A buckling coefficient, K_e, of about 0.8 is reasonable (see Appendix G of NDS) and is supported in the research literature on this topic for sheathed wall assemblies and studs with square-cut ends (i.e., not a pinned joint).

In cases where continuous support is not present (e.g., during construction), the designer may want to consider stability for both axes. Unsupported studs generally fail due to weak-axis buckling under a significantly lower load than would otherwise be possible with continuous lateral support in the weak-axis buckling direction.

Interior walls may be either load-bearing or nonload-bearing. Nonload-bearing interior walls are often called partitions (see Section 5.5.3). In either case, interior walls should be solidly fastened to the floor and ceiling framing and to the exterior wall framing where they abutt. It may be necessary to install extra studs, blocking, or nailers in the outside walls to provide for attachment of interior walls. The framing must also be arranged to provide a nailing surface for wallcovering materials at inside corners. For efficient construction details and concepts related to wall framing, refer to *Cost Effective Home Building: A Design and Construction Handbook* (NAHB, 1994).

Interior load-bearing walls typically support the floor or ceiling joists above when the clear span from exterior wall to exterior wall is greater than the spanning capability of the floor or ceiling joists. Interior walls, unlike exterior walls, seldom experience large transverse (i.e., out of plane) lateral loads; however, some building codes require interior walls to be designed for a minimum lateral load, such as 5 psf, for serviceability. If the interior wall is required only to resist axial loads, the designer may follow the design procedure demonstrated in Example 5.6 for the axial-load-only case. Generally, axial load design provides more-than-adequate resistance to a nominal lateral load.

If local code requirements do require wall studs to be designed to withstand a minimum lateral load, the designer should design load-bearing walls in accordance with the previous section on exterior load bearing walls. (Note that the load duration factor, C_D, of 1.6 is used for exterior load bearing walls when wind or earthquake loads are considered, whereas a load duration factor of 1.0 to 1.25 may be used for interior load-bearing walls and exterior walls analyzed for live and snow loads; refer to Section 5.2.4.1.)

5.5.3 NonLoad-Bearing Partitions

Interior partitions are not intended to support structural loads. Standard 2x4 or 2x3 wood stud interior partition walls are well proven in practice and do not require analysis. Openings within partitions do not require headers or trimmers and are commonly framed with single studs and horizontal members of the same size as the studs. Particularly in the case of closets, or other “tight” spaces, builders may frame certain partitions with smaller lumber, such as 2x2 studs or 2x4 studs turned flatwise to save space.

Where a minimum 5 psf lateral load check for serviceability is required in a nonload-bearing partition, the stud may be designed as a bending member or system similar to a simply supported floor joist, except that the only load is a 5 psf load uniformly distributed. The design approach and system factors in Sections 5.2 and 5.3 apply as appropriate.

5.5.4 Headers

Load-bearing headers are horizontal members that carry loads from a wall, ceiling, or floor or roof above and transfer the combined load to jack and king studs on each side of a window or door opening. The span of the header may be taken as the width of the rough opening measured between the jack studs supporting the ends of the header. Headers are usually built up from two nominal 2-inch-thick members.

Load-bearing header design and fabrication is similar to that for girders (see Section 5.4.3). This guide considers headers consisting of double members to be repetitive members; therefore, a repetitive member factor, C_r, of 1.1 to 1.2

should apply (refer to Table 5.4), along with a live load deflection limit of $\ell/240$ (refer to Table 5.6). Large openings or especially heavy loads may require stronger members such as engineered wood beams, hot-rolled steel, or flitch plate beams. Refer to *Cost-Effective Home Building: A Design and Construction Handbook* for economical framing solutions to reduce header loads and sizes (NAHB, 1994).

Headers are generally designed to support all loads from above; however, typical residential construction calls for a double top plate above the header. When an upper story is supported, a floor band joist and sole plate of the wall above are also spanning the wall opening below. These elements are all part of the resisting system. Recent header testing determined whether an adjustment factor (i.e., system factor or repetitive member factor) is justified in designing a header (HUD, 1999). The results showed that a repetitive member factor is valid for headers constructed of only two members as shown in Table 5.4 and that additional system effects produce large increases in capacity when the header is overlaid by a double top plate, band joist and sole plate as shown in Example 5.7. Consequently, an overall system factor of 1.8 was found to be a simple, conservative design solution. That system factor is applicable to the adjusted bending stress value, F_b', of the header member only. While this example covers only a very specific condition, it exemplifies the magnitude of potential system effect in similar conditions. In this case, the system effect is associated with load sharing and partial composite action. The above adjustment factor is not currently recognized in the NDS.

Refer to Table 5.8 for recommended allowable bending stress adjustment factors for use in the specific header design conditions related to the discussion above. For other conditions, refer to Table 5.4. Example 5.7 demonstrates the design approach for a typical header condition.

TABLE 5.8 ***Recommended System Adjustment Factors for Header Design***

Header Type and Application[1]	**Recommended C_r Value**[2]
2x10 double header of No. 2 Spruce-Pine-Fir	1.30[3]
Above header with double top plate, 2x10 floor band joist, and sole plate of wall located directly above.[4]	1.8

Notes:
[1]For other applications and lumber sizes or grades, refer to the C_r factors in Table 5.4 of Section 5.2.4.2.
[2]Apply C_r in lieu of Section 5.1.3 (Table 5.4) to determine adjusted allowable bending stress, F_b'.
[3]Use $C_r = 1.35$ when the header is overlaid by a minimum 2x4 double top plate without splices.
[4]Refer to Example 5.7 for an illustration of the header system.

Headers are not required in nonload-bearing walls. Openings can be framed with single studs and a horizontal header block of the same size. It is common practice to use a double 2x4 or triple 2x4 header for larger openings in nonload-bearing walls. In the interest of added rigidity and fastening surface, however, some builders use additional jamb studs for openings in nonload-bearing walls, but such studs are not required.

5.5.5 Columns

Columns are vertical members placed where an axial force is applied parallel to the longitudinal axis. Columns may fail by either crushing or buckling. Longer columns have a higher tendency than shorter columns to fail due to buckling. The load at which the column buckles (Euler buckling load) is directly related to the ratio of the column's unsupported length to its depth (slenderness factor). The equations provided in Section 5.3 are based on the NDS•3.7.1 provisions regarding the compression and stability of an axial compression member (i.e., column) and thus account for the slenderness factor.

Figure 5.6 illustrates three ways to construct columns using lumber. *Simple columns* are columns fabricated from a single piece of sawn lumber; *spaced columns* are fabricated from two or more individual members with their longitudinal axes parallel and separated with blocking at their ends and mid-point(s); *built-up columns* are solid columns fabricated from several individual members fastened together. Spaced columns as described in the NDS are not normally used in residential buildings and are not addressed here (refer to NDS•15.2 for the design of spaced columns).

Steel jack posts are also commonly used in residential construction; however, jack post manufacturers typically provide a rated capacity so that no design is required except the specification of the design load requirements and the selection of a suitable jack post that meets or exceeds the required loading. Typical 8-foot tall steel jack posts are made of pipe and have adjustable bases for floor leveling. The rated (design) capacity generally ranges from 10,000 to 20,000 lbs depending on the steel pipe diameter and wall thickness.

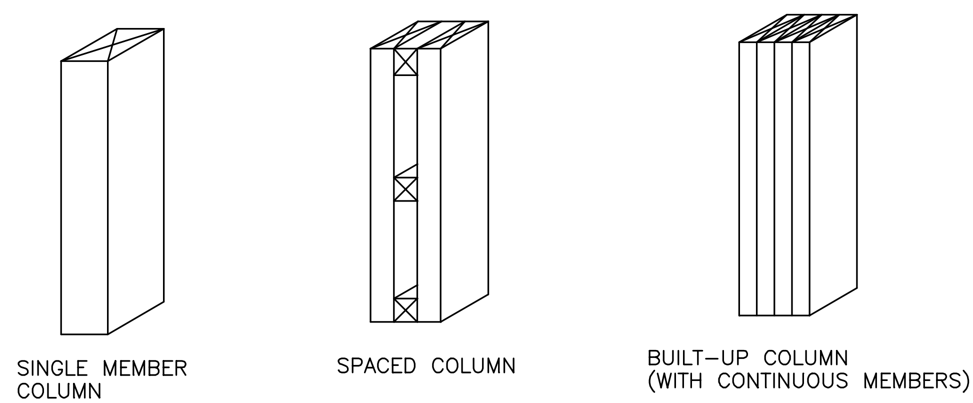
Simple columns are fabricated from one piece of sawn lumber. In residential construction, simple columns such as a 4x4 are common. The equations in Section 5.3 are used to design simple columns as demonstrated in Example 5.8.

Built-up columns are fabricated from several wood members fastened together with nails or bolts. They are commonly used in residential construction because smaller members can be easily fastened together at the jobsite to form a larger column with adequate capacity.

The nails or bolts used to connect the plys (i.e., the separate members) of a built-up column do not rigidly transfer shear loads; therefore, the bending load capacity of a built-up column is less than a single column of the same species, grade, and cross-sectional area when bending direction is perpendicular to the laminations (i.e., all members bending in their individual weak-axis direction). The coefficient, K_f, accounts for the capacity reduction in bending load in nailed or bolted built-up columns. It applies, however, only to the weak-axis buckling or bending direction of the individual members and therefore should not be used to determine C_p for column buckling in the strong-axis direction of the individual members. (Refer to NDS•15.3 for nailing and bolting requirements for built-up columns.)

The above consideration is not an issue when the built-up column is sufficiently braced in the weak-axis direction (i.e., embedded in a sheathed wall assembly). In this typical condition, the built-up column is actually stronger than a solid sawn member of equivalent size and grade because of the repetitive member effect on bending capacity (see Table 5.4). However, when the members in the built-up column are staggered or spliced, the column bending strength is reduced. While the NDS•15.3 provisions apply only to built-up columns with all members extending the full height of the column, design methods for spliced columns are available (ASAE, 1997).

FIGURE 5.6 ***Wood Column Types***



5.6 Roofs

The objectives of roof framing design are

- to support building dead and snow loads and to resist wind and seismic forces;
- to resist roof construction and maintenance loads;
- to provide a thermal and weather barrier;
- to provide support for interior ceiling finishes; and
- to provide attic space and access for electrical and mechanical equipment or storage.

5.6.1 General

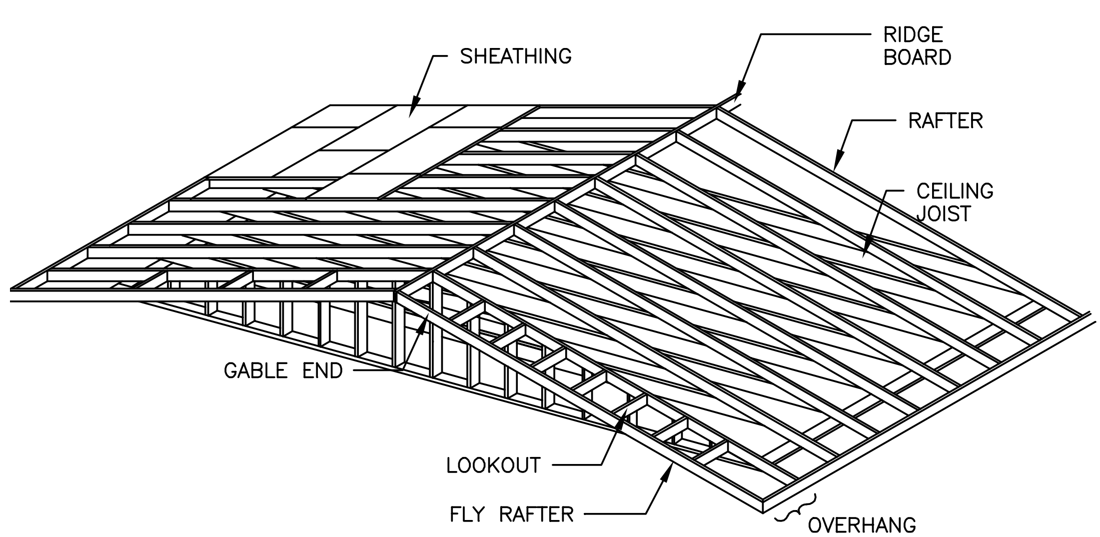
A roof in residential construction is typically a sloped structural system that supports gravity and lateral loads and transfers the loads to the walls below. Generally, the four options for wood roof construction are

- roof trusses;
- rafters and cross-ties;
- rafters with ridge beams (i.e. cathedral ceiling); and
- timber framing.

By far the most common types of residential roof construction use light-frame trusses, rafters, or a mix of these depending on roof layout. Figure 5.7 depicts conventional roof construction and roof framing elements. Rafters are repetitive framing members that support the roof sheathing and typically span from the exterior walls to a nonstructural ridge board (i.e., reaction plate). Rafter pairs may also be joined at the ridge with a gusset, thereby eliminating the need for a ridge board. Rafters may also be braced at or near mid-span using intermittent 2x vertical braces and a 2x runner crossing the bottom edges of the rafters. Ceiling joists are repetitive framing members that support ceiling and attic loads and transfer the loads to the walls and beams below. They are not normally designed to span between exterior walls and therefore require an intermediate bearing wall. Overhangs, where used, are framed extensions of the roof that extend beyond the exterior wall of the home, typically by 1 to 2 feet. Overhangs protect walls and windows from direct sun and rain and therefore offer durability and energy efficiency benefits.

Ceiling joists are typically connected to rafter pairs to resist outward thrust generated by loading on the roof. Where ceiling joists or cross-ties are eliminated to create a cathedral ceiling, a structural ridge beam must be used to support the roof at the ridge and to prevent outward thrust of the bearing walls. Ceiling joists and roof rafters are bending members that are designed similarly; therefore, this chapter groups them under one section.

FIGURE 5.7 ***Structural Elements of a Conventional Roof System***



Roof trusses are preengineered components. They are fabricated from 2-inch-thick dimension lumber connected with metal truss plates. They are generally more efficient than stick framing and are usually designed to span from exterior wall to exterior wall with no intermediate support. In more complex portions of roof systems, it is still common to use rafter framing techniques.

Roof sheathing is a thin structural element, usually plywood or oriented strand board, that supports roof loads and distributes lateral and axial loads to the roof framing system. Roof sheathing also provides lateral support to the roof framing members and serves as a membrane or diaphragm to resist and distribute lateral building loads from wind or earthquakes (refer to Chapter 6).

Roof systems are designed to withstand dead, live, snow, and wind uplift loads; in addition, they are designed to withstand lateral loads, such as wind and earthquake loads, transverse to the roof system. The design procedure discussed herein addresses dimension lumber roof systems designed according to the NDS. Where appropriate, the procedure incorporates modifications of the NDS. Section 5.3 summarizes the general design equations and design checks based on the NDS. Refer to Chapter 6 for the design of roofs with respect to lateral loads on the overall structure; refer to Chapter 7 for guidance on the design of connections.

When designing roof elements or components, the designer needs to consider the following load combinations from Chapter 3 (Table 3.1):

- $D + (L_r \text{ or } S)$
- $0.6\ D + W_u$
- $D + W$

The following sections refer to the span of the member. The NDS defines span as the clear span of the member plus one-half the required bearing at each end of the member. For simplicity, the clear span between bearing points is used herein.

Finally, roofs exhibit system behavior that is in many respects similar to floor framing (see Section 5.4); however, sloped roofs also exhibit unique system behavior. For example, the sheathing membrane or diaphragm on a sloped roof acts as a folded plate that helps resist gravity loads. The effect of the folded plate becomes more pronounced as roof pitch becomes steeper. Such a system effect is usually not considered in design but explains why light wood-framed roof systems may resist loads several times greater than their design capacity. Recent research on trussed roof assemblies with wood structural panel sheathing points to a system capacity increase factor of 1.1 to 1.5 relative to the design of an individual truss (Wolfe and LaBissoniere, 1991; Wolfe, 1996; Mtenga, 1998). Thus, a conservative system factor of 1.15 is recommended in this document for chord bending stresses and a factor of 1.1 for chord tension and compression stresses.

5.6.2 Conventional Roof Framing

This section addresses the design of conventional roof rafters, ceiling joists (cross-ties), ridge beams, and hip and valley rafters. The design procedure for a rafter and ceiling joist system is similar to that of a truss, except that the assembly of components and connections is site-built. It is common practice to use a standard pin-joint analysis to determine axial forces in the members and shear forces at their connections. The ceiling joists and rafters are then usually sized according to their individual applied bending loads taking into account that the axial load effects on the members themselves can be dismissed by judgment based on the large system effects in sheathed roof construction. Frequently, intermediate rafter braces that are similar to truss web members are also used. Standard construction details and span tables for rafters and ceiling joists can be found in the *International One- and Two-Family Dwelling Code* (ICC, 1998). These tables generally provide allowable horizontal rafter span with disregard to any difference that roof slope may have on axial and bending loads experienced in the rafters. This approach is generally considered as standard practice. Example 5.9 demonstrates two design approaches for a simply-supported, sloped rafter as illustrated in Figure 5.8.

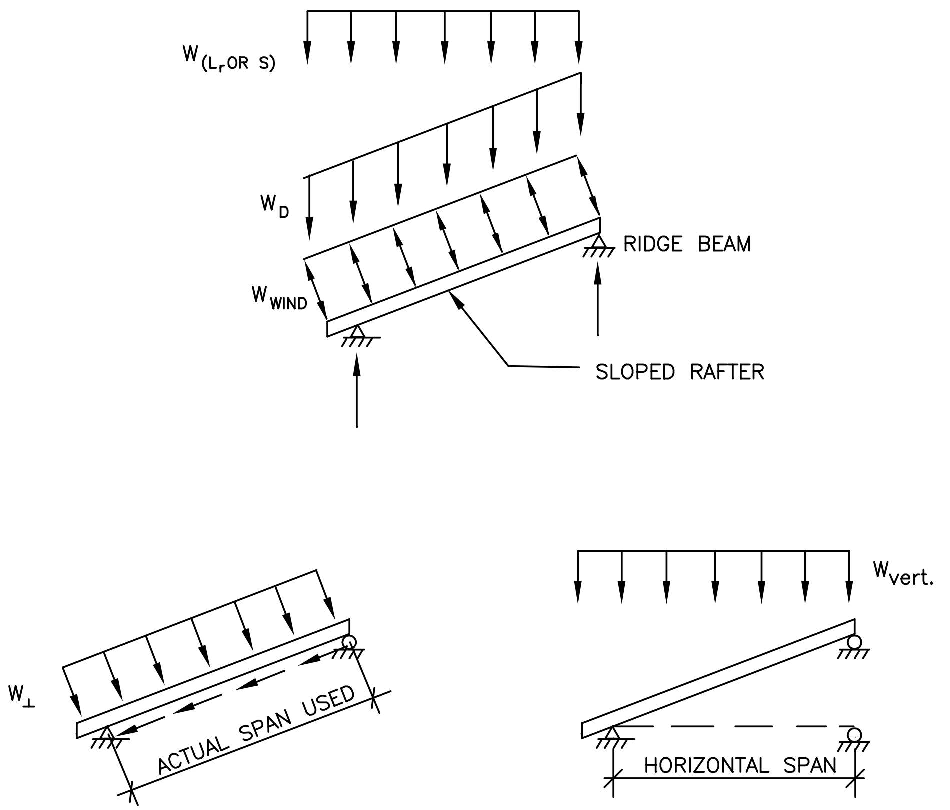
Structural ridge beams are designed to support roof rafters at the ridge when there are no ceiling joists or cross-ties to resist the outward thrust of rafters that would otherwise occur. A repetitive member factor, C_r, is applicable if the ridge beam is composed of two or more members (see Table 5.4). It should also be noted that any additional roof system benefit, such as the folded plate action of the roof sheathing diaphragm, goes ignored in its structural contribution to the ridge beam, particularly for steep-sloped roofs. Example 5.10 demonstrates the design approach for ridge beams.

Roofs with hips and valleys are constructed with rafters framed into a hip or valley rafter as appropriate and, in practice, are typically one to two sizes larger than the rafters they support, e.g., 2x8 or 2x10 hip for 2x6 rafters. While hip and valley rafters experience a unique tributary load pattern or area, they are generally designed much like ridge beams. The folded plate effect of the roof sheathing diaphragm provides support to a hip or valley rafter in a manner similar to that discussed for ridge beams. However, beneficial system effect generally goes ignored because of the lack of definitive technical guidance. Nonetheless, the use of design judgment should not be ruled out. Example 5.11 demonstrates the design of a hip rafter.

5.6.3 Roof Trusses

Roof trusses incorporate rafters (top chords) and ceiling joists (bottom chords) into a structural frame fabricated from 2-inch-thick dimension lumber, usually 2x4s or 2x6s. A combination of web members are positioned between the top and bottom chords, usually in triangular arrangements that form a rigid framework. Many different truss configurations are possible, including open trusses for attic rooms and cathedral or scissor trusses with sloped top and bottom chords. The wood truss members are connected by metal truss plates punched with barbs (i.e., teeth) that are pressed into the truss members. Roof trusses are able to span the entire width of a home without interior support walls, allowing complete freedom in partitioning interior living space. The *Metal Plate Connected Wood Truss Handbook* contains span tables for typical truss designs (WTCA, 1997).

FIGURE 5.8 Design Methods and Assumptions for a Sloped Roof Rafter



METHOD A: RESOLVE LOADS TO COMPONENT PERPENDICULAR TO SLOPED RAFTER (THEORETICALLY CORRECT APPROACH)

METHOD B: RESOLVE LOADS TO VERTICAL COMPONENTS AND APPLY TO HORIZONTAL SPAN (COMMON PRACTICE)

NOTE: SNOW AND LIVE LOADS IN CODES ARE GENERALLY CONSIDERED RELATIVE TO THE HORIZONTAL AREA. WIND LOADS ACT PERPENDICULAR TO THE BUILDING SURFACE. DEAD LOAD IS A VERTICAL LOAD AND APPLIES TO THE ACTUAL AREA/LENGTH OF THE MEMBER AND SUPPORTED COMPONENTS.

Roof truss manufacturers normally provide the required engineering design based on the loading conditions specified by the building designer. The building designer is responsible for providing the following items to the truss manufacturer for design:

- design loads;
- truss profile;
- support locations; and
- any special requirements.

The building designer should also provide for permanent bracing of the truss system at locations designated by the truss designer. In general, such bracing may involve vertical cross-bracing, runners on the bottom chord, and bracing of certain web members. In typical light-frame residential roof construction, properly attached roof sheathing provides adequate overall bracing of the roof truss system and ceiling finishes normally provide lateral support to the bottom chord of the truss. The only exception is long web members that may experience buckling from excessive compressive loads. Gable endwall bracing is discussed separately in Section 5.6.6 as it pertains to the role of the roof system in supporting the walls against lateral loads, particularly those produced by wind. For more information and details on permanent bracing of trusses, refer to *Commentary for Permanent Bracing of Metal Plate Connected Wood Trusses* (WTCA, 1999). Temporary bracing during construction is usually the responsibility of the contractor and is important for worker safety. For additional guidance on temporary bracing, consult the *Metal Plate Connected Wood Truss Handbook* pages 14-1 through 15-12 and Appendix L (WTCA, 1997). For additional guidance on roles and responsibilities, refer to *Standard Practice for Metal Plate Connected Wood Truss Design Responsibilities* (WTCA, 1995).

The National Design Standard for Metal Plate Connected Wood Truss Construction (ANSI/TPI 1-95) governs the design of trusses. Available from the Truss Plate Institute (TPI, 1995a and b), ANSI/TPI 1-95 includes the structural design procedure as well as requirements for truss installation and bracing and standards for the manufacture of metal plate connectors. A computer program, PPSA, is also available for a detailed finite element analysis (Triche and Suddarth, 1993). Truss plate manufacturers and truss fabricators generally have proprietary computerized design software based on ANSI/TPI 1-95, with modifications tailored to their particular truss-plate characteristics.

The designer should note that cracking and separation of ceiling finishes may occur at joints between the walls and ceiling of roofs. In the unfavorable condition of high attic humidity, the top chord of a truss may expand while the lower roof members, typically buried under attic insulation, may not be similarly affected. Thus, a truss may bow upward slightly. Other factors that commonly cause interior finish cracking are not in any way associated with the roof truss, including shrinkage of floor framing members, foundation settlement, or heavy loading of a long-span floor resulting in excessive deflection that may “pull” a partition wall downward from its attachment at the ceiling. To reduce the potential for cracking of ceiling finishes at partition wall intersections, 2x wood blocking should be installed at the top of partition wall plates as a backer for the ceiling finish material (i.e., gypsum board). Ceiling drywall should not be fastened to the blocking or to the truss bottom chord within 16 to 24 inches of the partition. Proprietary clips are available for use in place of wood blocking and resilient metal “hat” channels may also be used to attach the ceiling finish to the roof framing. Details that show how to minimize partition-ceiling separation problems can be found on the WTCA website at (www.woodtruss.com) or by contacting WTCA to obtain a “Partition Separation” brochure.

Trusses are also frequently used for floor construction to obtain long spans and to allow for the placement of mechanical systems (i.e., ductwork and sanitary drains) in the floor cavity. In addition, trusses have been used to provide a complete house frame (NAHBRC, 1982). One efficient use of a roof truss is as a structural truss for the gable end above a garage opening to effectively eliminate the need for a garage door header. For other efficient framing design concepts and ideas, refer to *Cost Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994).

5.6.4 Roof Sheathing

Roof sheathing thickness is typically governed by the spacing of roof framing members and live or snow loads. Sheathing is normally in accordance with prescriptive sheathing span rating tables published in a building code or made available by manufacturers. If the limit of the prescriptive tables is exceeded, the designer may need to perform calculations; however, such calculations are rarely necessary in residential construction. The process of selecting rated roof sheathing is similar to that for floor sheathing in Example 5.5.

The fasteners used to attach sheathing to roof rafters are primarily nails. The most popular nail types are sinker, box, and common, of which all have different characteristics that affect structural properties (refer to Chapter 7). Proprietary power-driven fasteners (i.e., pneumatic nails and staples) are also used extensively. The building codes and APA tables recommend a fastener schedule for connecting sheathing to roof rafters. Generally, nails are placed at a minimum 6 inches on center at edges and 12 inches on center at intermediate supports. A 6-inch fastener spacing should also be used at the gable-end framing to help brace the gable-end. Nail size is typically 8d, particularly since thinner power driven nails are most commonly used. Roof sheathing is commonly 7/16- to 5/8-inch-thick on residential roofs. Note that in some cases shear loads in the roof diaphragm resulting from lateral loads (i.e., wind and earthquake) may require a more stringent fastening schedule; refer to Chapter 6 for a discussion of fastening schedules for lateral load design. More importantly, large suction pressures on roof sheathing in high wind areas (see Chapter 3) will require a larger fastener and/or closer spacing. In hurricane-prone regions, it is common to require an 8d deformed shank nail with a 6 inch on center spacing at all framing connections. At the gable end truss or rafter, a 4 inch spacing is common.

5.6.5 Roof Overhangs

Overhangs are projections of the roof system beyond the exterior wall line at either the eave or the rake (the sloped gable end). Overhangs protect walls from rain and shade windows from direct sun. When a roof is framed with wood trusses, an eave overhang is typically constructed by extending the top chord beyond the exterior wall. When a roof is framed with rafters, the eave overhang is constructed by using rafters that extend beyond the exterior wall. The rafters are cut with a “bird-mouth” to conform to the bearing support. Gable end overhangs are usually framed by using a ladder panel that cantilevers over the gable end for either stick-framed or truss roofs. Refer to Figure 5.9 for illustrations of various overhang constructions.

A study completed in 1978 by the Southern Forest Experiment Station for the U.S. Department of Housing and Urban Development found that the protection afforded by overhangs extends the life of the wall below, particularly if the wall is constructed of wood materials (HUD, 1978). Entitled the *Prevention and Control of Decay in Homes*, the report correlates the climate index of a geographic area with a suggested overhang width and recommends highly conservative widths. As a reasonable guideline (given that in many cases no overhang is provided), protective overhang widths should be 12 to 24 inches in moist, humid climates and more if practicable. A reasonable rule-of-thumb to apply is to provide a minimum of 12 inches of overhang width for each story of protected wall below. However, overhang width can significantly increase wind uplift loads on a roof, particularly in high wind regions. The detailing of overhang framing connections (particularly at the rake overhang on a gable end) is a critical consideration in hurricane-prone regions. Often, standard metal clips or straps provide adequate connection. The need for special rake overhang design detailing depends on the length of the overhang, the design wind load condition, and the framing technique that supports the overhang (i.e., 2x outriggers versus cantilevered roof sheathing supporting ladder overhang framing).

5.6.6 Gable-End Wall Bracing

Roof framing provides lateral support to the top of the walls where trusses and rafters are attached to the wall top plate. Likewise, floor framing provides lateral support to the top and bottom of walls, including the top of foundation walls. At a gable end, however, the top of the wall is not directly connected to roof framing members; instead, it is attached to the bottom of a gable-end truss and lateral support at the top of the wall is provided by the ceiling diaphragm. In higher-wind regions, the joint may become a “hinge” if the ceiling diaphragm becomes overloaded. Accordingly, it is common practice to brace the top of the end wall (or bottom of the gable end roof framing) with 2x4 or 2x6 framing members that slope upward to the roof diaphragm to attach to a blocking or a ridge “beam” as shown in Figure 5.9. Alternatively, braces may be laid flatwise on ceiling joists or truss bottom chords and angled to the walls that are perpendicular to the gable-end wall. Given that braces must transfer inward and outward forces resulting from positive wind pressure or suction on the gable-end wall, they are commonly attached to the top of the gable-end wall with straps to transfer tension forces that may develop in hurricanes and other extreme wind conditions. The need for and special detailing of gable-end wall braces depends on the height and area of the gable end (i.e., tributary area) and the design wind load. The gable endwall can also be braced by the use of a wood structural panel attached to the gable end framing and the ceiling framing members.

As an alternative to the above strategy, the gable-end wall may be framed with continuous studs that extend to the roof sheathing at the gable end (i.e., balloon-framed). If the gable-end wall encloses a two-story room–such as a room with a cathedral ceiling, it is especially important that the studs extend to the roof sheathing; otherwise, a hinge may develop in the wall and cause cracking of wall finishes (even in a moderate wind) and could easily precipitate failure of the wall in an extreme wind. Depending on wall height, stud size, stud spacing, and the design wind load condition, taller, full-height studs may need to be increased in size to meet deflection or bending capacity requirements. Some designer judgment should be exercised in this framing application with respect to the application of deflection criteria. The system deflection adjustment factors of

FIGURE 5.9 Typical Roof Overhang Construction

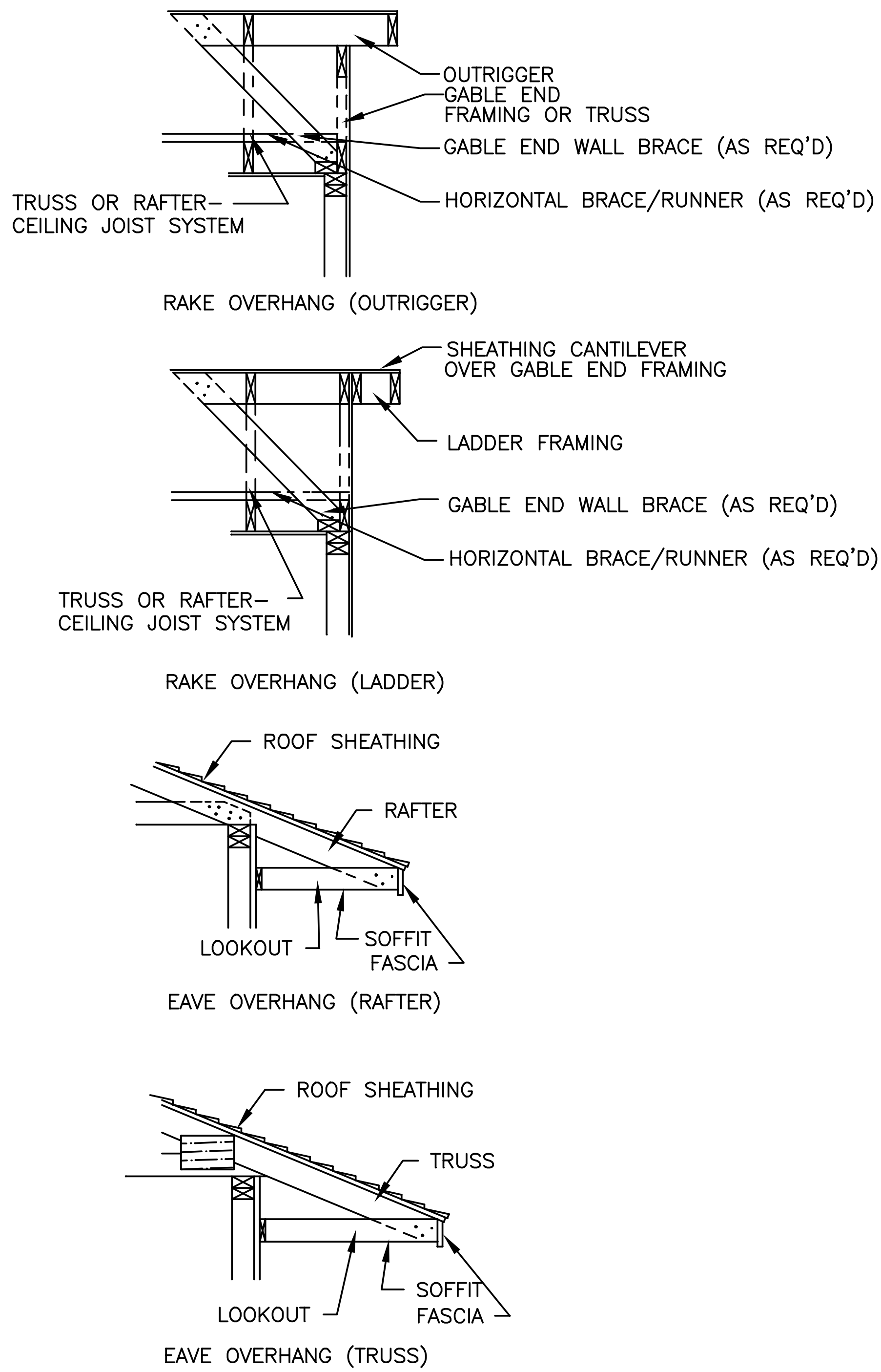


Table 5.6 may assist in dealing with the need to meet a reasonable serviceability limit for deflection (see Section 5.3.2).

Finally, as an alternative that avoids the gable-end wall bracing problem, a hip roof may be used. The hip shape is inherently more resistant to wind damage in hurricane-prone wind environments (see Chapter 1) and braces the end walls against lateral wind loads by direct attachment to rafters.

5.7 Design Examples

In this section, a number of design examples illustrate the design of various elements discussed in this chapter. The examples are intended to also provide practical advice. Therefore, the examples are embellished with numerous notes and recommendations to improve the practicality and function of various possible design solutions. They are also intended to promote the designer's creativity in arriving at the best possible solution for a particular application

EXAMPLE 5.1 *Typical Simple Span Floor Joist Design*

Given

Live load (L)	=	30 psf (bedroom area)
Dead load (D)	=	10 psf
Trial joist spacing	=	16 on center
Trial joist size	=	2x8
Trial joist species and grade	=	Hem-Fir, No. 1 (S-dry, 19% MC)

Find Maximum span for specified joist member.

Solution

1. Determine tabulated design values by using NDS-S (Tables 4A and 1B)

F_b = 975 psi $\qquad I_{xx}$ = 47.63 in^4

F_v = 75 psi $\qquad S_{xx}$ = 13.14 in^3

$F_{c\perp}$ = 405 psi $\qquad b$ = 1.5 in

E = 1,500,000 psi $\qquad d$ = 7.25 in

2. Lumber property adjustments and adjusted design values (Section 5.2.4 and NDS•2.3)

C_D = 1.0 (Section 5.2.4.1)

C_r = 1.15 (Table 5.4)

C_F = 1.2 (NDS-S Table 4A adjustment factors)

C_H = 2.0 (Section 5.2.4.3)

C_L = 1.0 (NDS•3.3.3, continuous lateral support)

C_b = 1.0 (NDS•2.3.10)

$F_b' = F_bC_rC_FC_DC_L$ = 975 (1.15)(1.2)(1.0)(1.0) = 1,345 psi

$F_v' = F_vC_HC_D$ = 75 (2)(1.0) = 150 psi

$F_{c\perp}' = F_{c\perp}C_b$ = 405 (1.0) = 405 psi

$E' = E$ = 1,500,000 psi

3. Calculate the applied load

W = (joist spacing)(D+L) = (16 in)(1 ft/12 in)(40 psf) = 53.3 plf

4. Determine maximum clear span based on bending capacity

$$M_{max} = \frac{w\ell^2}{8} = \frac{(53.3\text{ plf})(\ell^2)}{8} = 6.66\,\ell^2$$

$$f_b = \frac{M}{S} = \frac{(6.66\ell^2)(12\,{}^{in}\!/_{ft})}{13.14\text{ in}^3} = 6.08\,\ell^2$$

$f_b \leq F_b'$

$6.08\,\ell^2 \leq$ 1,345 psi

ℓ^2 = 221

ℓ = 14.9 ft = 14 ft-11 in (maximum clear span due to bending stress)

5. Determine maximum clear span based on horizontal shear capacity

$$V_{max} = \frac{w\ell}{2} = \frac{(53.3\ \text{plf})(\ell)}{2} = 26.7\ \ell$$

$$f_v = \frac{3V}{2A} = \frac{3}{2}\left(\frac{26.7\,\ell}{(1.5\ \text{in})(7.25\ \text{in})}\right) = 3.7\,\ell$$

$f_v \leq F_v'$

$3.7\ell \leq 150$ psi

ℓ = 40.5 ft = 40 ft-6 in (maximum clear span due to horizontal shear stress)

6. Determine maximum clear span based on bearing capacity

Bearing length = (3.5-in top plate width) - (1.5-in rim joist width) = 2 in

$$f_{c\perp} = \frac{\frac{1}{2}w\ell}{A_b} = \frac{\frac{1}{2}(53.3\ \text{plf})(\ell)}{(2\ \text{in})(1.5\ \text{in})} = 8.9\,\ell$$

$f_{c\perp} < F_{c\perp}'$

$8.9\ell \leq 405$ psi

ℓ = 45.5 ft = 45 ft-6 in (maximum clear span due to bearing stress)

7. Consider maximum clear span based on deflection criteria (Section 5.3.2)

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(40\ \text{plf})^*(\ell)^4\ (1{,}728\ \text{in}^3/\text{ft}^3)}{384(1{,}500{,}000\ \text{psi})(47.63\ \text{in}^4)} = 1.26 \times 10^{-5}\ell^4$$

*applied live load of 30 psf only

$$\rho_{all} = \frac{\ell}{360}(12\ \text{in/ft}) = 0.033\,\ell$$

$\rho_{max} \leq \rho_{all}$

$1.26 \times 10^{-5}\,\ell^4 \leq 0.033\,\ell$

$\ell^3 = 2{,}619$

ℓ = 13.8 ft = 13 ft-10 in (recommended clear span limit due to deflection criteria)

8. Consider floor vibration (Section 5.3.2)

The serviceability deflection check was based on the design floor live load for bedroom areas of 30 psf. The vibration control recommended in Section 5.3.2 recommends using a 40 psf design floor live load with the ℓ/360 deflection limit. Given that the span will not be greater than 15 feet, it is not necessary to use the absolute deflection limit of 0.5 inch.

w = (16 in)(1 ft/12 in)(40 psf) = 53.3 plf

$$\rho_{all} = (\ell/360)(12\ \text{in/ft}) = 0.033\,\ell$$

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(53.3\ \text{plf})^*(\ell^4)\ (1{,}728\ \text{in}^3/\text{ft}^3)}{384(1.5 \times 10^6\ \text{psi})(47.63\ \text{in}^4)} = 1.7 \times 10^{-5}\,\ell^4$$

*applied live load of 40 psf only

$\rho_{max} \leq \rho_{all}$

$1.7 \times 10^{-5}\,\ell^4 \leq 0.033\,\ell$

$\ell^3 = 1{,}941$

ℓ = 12.5 ft = 12 ft-6 in (recommended clear span limit due to vibration)

Conclusion

The serviceability limit states used for deflection and floor vibration limit the maximum span. The deflection limited span is 13 ft-10 in and the vibration limited span is 12 ft-6 in. Span selection based on deflection or vibration is an issue of designer judgment. The maximum span limited by the structural safety checks was 14 ft-11 in due to bending. Therefore, the serviceability limit will provide a notable safety margin above that required. Thus, No. 2 grade lumber should be considered for economy in that it will have only a small effect on the serviceability limits. Conversely, if floor stiffness is not an expected issue with the owner or occupant, the span may be increased beyond the serviceability limits if needed to “make it work.” Many serviceable homes have been built with 2x8 floor joists spanning as much as 15 feet; however, if occupants have a low tolerance for floor vibration, a lesser span should be considered.

For instructional reasons, shrinkage across the depth of the floor joist or floor system may be estimated as follows based on the equations in Section 5.3.2:

d_1 = 7.25 in $\quad$ M_1 = 19% maximum (S-dry lumber)

d_2 = ? $\quad$ M_2 = 10% (estimated equilibrium MC)

$$d_2 = d_1 \left(\frac{1 - \frac{a - 0.2M_2}{100}}{1 - \frac{a - 0.2M_1}{100}} \right) = 7.25 \text{ in} \left(\frac{1 - \frac{6.031 - 0.2(10)}{100}}{1 - \frac{6.031 - 0.2(19)}{100}} \right) = 7.1 \text{ in}$$

Shrinkage ≅ 7.25 ft-7.08 in = 0.15 in (almost 3/16 in)

In a typical wood-framed house, shrinkage should not be a problem, provided that it is uniform throughout the floor system. In multistory platform frame construction, the same amount of shrinkage across each floor can add up to become a problem, and mechanical systems and structural details should allow for such movement. Kiln-dried lumber may be specified to limit shrinkage and building movement after construction.

EXAMPLE 5.2 *Simple Span Floor Joist Design (Optimize Lumber)*

Given

Live load (L) = 40 psf
Dead load (D) = 10 psf
Clear span = 14 ft-2 in
Joist size = 2x10

Find Optimum lumber species and grade

Solution

1. Calculate the applied load

W = (joist spacing)(D+L) = (2 ft)(40 psf +10 psf) = 100 plf

2. Determine bending stress

$$M_{max} = \frac{w\ell^2}{8} = \frac{(100\ \text{plf})(14.17\ \text{ft})^2}{8} = 2{,}510\ \text{ft-lb}$$

$$F_b = \frac{M}{S} = \frac{(2{,}510\ \text{ft}-\text{lb})(12\ \text{in/ft})}{21.39\ \text{in}^3} = 1{,}408\ \text{psi}$$

3. Determine horizontal shear stress

$$V_{max} = \frac{w\ell}{2} = \frac{(100\ \text{plf})(14.17\ \text{ft})}{2} = 709\ \text{lb}$$

$$f_v = \frac{3V}{2A} = \frac{3(709\ \text{lb})}{2(1.5\ \text{in})(9.25\ \text{in})} = 77\ \text{psi}$$

4. Determine bearing stress:

$$R_1 = R_2 = V_{max} = 709\ \text{lb}$$

$$f_{c\perp} = \frac{R}{A_b} = \frac{709\ \text{lb}}{(2\ \text{in})(1.5\ \text{in})} = 236\ \text{psi}$$

Wall and roof loads, if any, are carried through rim/band joist

5. Determine minimum modulus of elasticity due to selected deflection criteria

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(80\ \text{plf})^*(14.17\ \text{ft})^4(1{,}728\ \text{in}^3/\text{ft}^3)}{384E\ (98.93\ \text{in}^4)} = 733{,}540/E$$

*includes live load of 40 psf only

$$\rho_{all} \leq \ell/360$$

$$\rho_{max} \leq \rho_{all}$$

$$\frac{733{,}540}{E} = \frac{(14.17\ \text{ft})(12\ \text{in/ft})}{360}$$

$$E_{min} = 1.55 \times 10^6\ \text{psi}$$

6. Determine minimum modulus of elasticity due to vibration

The span required is not greater than 15 feet and the ℓ/360 deflection check uses a 40 psf floor live load. Therefore, the deflection check is assumed to provide adequate vibration control.

7. Determine minimum required unadjusted properties by using NDS tabulated lumber data

Bending

$$f_b \leq F_b'$$
$$F_b' = F_b C_r C_F C_D$$
$$F_{bmin} = \frac{f_b}{C_r C_F C_D} = \frac{1{,}408 \text{ psi}}{(1.15)(1.1)(1.0)} = 1{,}113 \text{ psi}$$

Horizontal shear

$$f_v \leq F_v'$$
$$F_v' = F_v C_H C_D$$
$$F_{vmin} = \frac{f_v}{C_H C_D} = \frac{77 \text{ psi}}{(2)(1.0)} = 39 \text{ psi}$$

Bearing

$$f_{c\perp} \leq F_{c\perp}' \quad \text{(assume minimum 2-in bearing)}$$
$$F_{c\perp} = F_{c\perp} C_b$$
$$F_{c\perp min} = \frac{f_{c\perp}}{(1.0)} = 236 \text{ psi}$$

Minimum unadjusted tabulated properties required

F_b = 1,113 psi $F_{c\perp}$ = 236 psi
F_v = 39 psi E = 1.55×10^6 psi

8. Select optimum lumber grade considering local availability and price by using NDS-S Table 4A or 4B data

Minimum No. 2 grade lumber is recommended for floor joists because of factors related to lumber quality such as potential warping and straightness that may affect constructability and create call-backs.

Considering 2x10 Douglas Fir-Larch, the grade below (No. 1 and Btr) was selected to meet the required properties.

F_b	=	1,200 psi	>	1,113 psi	OK
F_v	=	95 psi	>	39 psi	OK
$F_{c\perp}$	=	625 psi	>	236 psi	OK
E	=	1.8×10^6 psi	>	1.55×10^6 psi	OK

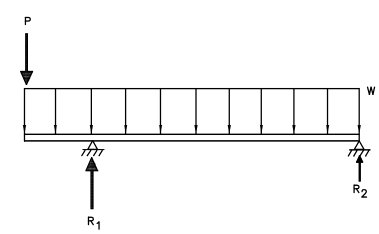
Conclusion

Many other species and grades should be considered depending on local availability and cost. Also, the No. 1 and higher grades are generally considered as "premium" lumber. A more economical design may be possible by using a closer joist spacing to allow for a lower grade (i.e, 19.2 inches on center or 16 inches on center). Also, a lower grade 2x12 should be considered or, perhaps, engineered wood I-joists.

EXAMPLE 5.3 *Cantilevered Floor Joist*

Given

Joist spacing = 16 in on center
Joist size = 2x10
Bearing length = 3-1/2 in
Species = Douglas Fir-Larch, No.1 Grade
Loads on cantilever joist (see Chapter 3)
Floor live load (L) = 40 psf
Floor dead load (D) = 10 psf
Loads for concentrated load at end of cantilever (see Chapter 3)
Roof snow load (S) = 11 psf (15 psf ground snow load and 7:12 roof pitch)
Roof dead load (D) = 12 psf
Wall dead load (D) = 8 psf
Roof span = 28 ft (clear span plus 1 ft overhang)
Wall height = 8 ft



Cantilever Joist Load Diagram

Find

Determine the maximum cantilever span for the specified floor joist based on these load combinations (Chapter 3, Table 3.1):

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

The analysis does not consider wind uplift that may control connections in high-wind areas, but not necessarily the cantilever joist selection.

Deflection at the end of the cantilever should be based on a limit appropriate to the given application. The application differs from a normal concern with mid-span deflection; experience indicates that deflection limits can be safely and serviceably relaxed in the present application. A deflection limit of ℓ/120 inches at the end of cantilever is recommended, particularly when the partial composite action of the sheathing is neglected in determining the moment of inertia, I, for the deflection analysis.

Solution

1. Determine tabulated design values for species and grade from the NDS-S

F_b =	1000 psi	S =	21.39 in^3
F_v =	95 psi	I =	98.93 in^4
$F_{c\perp}$ =	625 psi	b =	1.5 in
E =	$1.7x10^6$ psi	d =	9.25 in

2. Determine lumber property adjustments (see Section 5.2.4)

C_r =	1.15	C_F =	1.1
C_H =	2.0	C_D =	1.25 (includes snow)
C_b^* =	1.11	C_L =	1.0 (continuous lateral support)**

*Joist bearing not at end of member (see NDS•2.3.10)

**The bottom (compression edge) of the cantilever is assuemd to be laterally braced with wood structural panel sheathing or equivalent. If not, the value of CL is dependent on the slenderness ratio (see NDS•3.3.3).

$F_b' = F_bC_rC_FC_DC_L$ = (1000 psi)(1.15)(1.1)(1.25)(1.0) = 1,581 psi

$F_v' = F_vC_HC_D$ = (95)(2)(1.25) = 238 psi

$F_{c\perp}' = F_{c\perp}C_b$ = 625 (1.11) = 694 psi

$E' = E = 1.7x10^6$ psi

3. Determine design loads on cantilever joist

The following load combinations (based on Chapter 3, Table 3.1) will be investigated for several load cases that may govern different safety or serviceability checks

Case I: D+S - Cantilever Deflection Check
- P = wall and roof load (lb) at end of cantilever = *f* (D+S)
- w = uniform load (plf) on joist = *f* (D only)

Case II: D+L - Deflection at Interior Span
- P = *f* (D only)
- w = *f* (D+L)

Case III: D+S+0.3L or D+L+0.3S - Bending and Horizontal Shear at Exterior Bearing Support

a. P = *f* (D+S)
   w = *f* (D + 0.3L)

b. P = *f* (D+0.3S)
   w = *f* (D+L)

The following values of P and W are determined by using the nominal design loads, roof span, wall height, and joist spacing given above

	Case I	Case II	Case IIIa	Case IIIb
P =	544 lb	325 lb	544 lb	390 lb
W =	13.3 plf	66.5 plf	29.3 plf	66.5 lb

Inspection of these loading conditions confirms that Case I controls deflection at the end of the cantilever, Case II controls deflection in the interior span, and either Case IIIa or IIIb controls the structural safety checks (i.e., bending, horizontal shear, and bearing).

Since the cantilever span, *X*, is unknown at this point, it is not possible to determine structural actions in the joist (i.e., shear and moment) by using traditional engineering mechanics and free-body diagrams. However, the beam equations could be solved and a solution for *X* iterated for all required structural safety and serviceability checks (by computer). Therefore, a trial value for *X* is determined in the next step. If an off-the-shelf computer program is used, verify its method of evaluating the above load cases.

4. Determine a trial cantilever span based on a deflection limit of $\ell/120$ and load Case I.

Use a 2 ft-10 in cantilever span (calculations not shown - see beam equations in Appendix A).

5. Determine the maximum bending moment and shear for the three load cases governing the structural safety design checks by using the trail cantilever span:

The following is determined by using free-body diagrams and shear and moment diagrams (or beam equations, see Appendix A)

	Case II	Case IIIa	Case IIIb
R_1	1,008 lb	938 lb	**1,088 lb**
R_2	301 lb	40 lb	286 lb
V_{max}*	511 lb	**626 lb**	576 lb
M_{max}	1,170 ft-lb	**1,638 ft-lb**	1,352 lb

**NDS•3.4.3 allows loads within a distance of the member depth, d, from the bearing support to be ignored in the calculation of shear V when checking horizontal shear stress. However, this portion of the load must be included in an analysis of the bending moment. It would reduce the value of V_{max} as calculated above by using beam equations by approximately 100 pounds in Case II and Case IIIb and about 44 pounds in Case IIIa by eliminating the uniform load, w, within a distance, d, from the exterior bearing support.*

6. Determine design bending moment capacity of the given joist and verify adequacy

$$F_b' \geq f_b = \frac{M_{all}}{S}$$

$$M_{all} = F_b'S = (1{,}581\ \text{psi})(21.4\ \text{in}^3)(1\ \text{ft}/12\ \text{in})$$

$$= 2{,}819\ \text{ft-lb}$$

$$M_{all} > M_{max} = 1{,}638\ \text{ft-lb} \qquad \text{OK}$$

7. Determine design shear capacity of the given joist and verify adequacy:

$$F_v = \frac{3V_{all}}{2A} \text{ and } F_v \geq F_v'$$

$$V_{all} = \frac{2AF_v'}{3} = \frac{2(1.5\ \text{in})(9.25\ \text{in})(238\ \text{psi})}{3}$$

$$= 2{,}202\ \text{lbs}$$

$$V_{all} > V_{max} = 626\ \text{lbs} \quad \text{OK}$$

8. Check bearing stress

$$f_{c\perp} = \frac{R_{max}}{A_b} = \frac{1{,}088 \text{ lb}}{(1.5 \text{ in})(3.5 \text{ in})}$$

$$= 207 \text{ psi}$$

$$F_{c\perp}' = 694 \text{ psi} > 207 \text{ psi} \quad \text{OK}$$

Conclusion

A cantilever span of 2 ft-10 in (2.8 feet) is structurally adequate. The span is controlled by the selected deflection limit (i.e., serviceability) which illustrates the significance of using judgment when establishing and evaluating serviceability criteria. Allowance for a 2-foot cantilever is a common field practice in standard simple span joist tables for conventional residential construction. A check regarding interior span deflection of the joist using load Case II may be appropriate if floor vibration is a concern. However, unacceptable vibration is unlikely given that the span is only 12 feet. Also, Douglas-Fir, Larch, No. 1 Grade, is considered premium framing lumber and No. 2 Grade member should be evaluated, particularly if only a 2-foot cantilever is required.

EXAMPLE 5.4 *Built-Up Floor Girder Design*

Given

Loads

Floor live load	=	40 psf
Floor dead load	=	10 psf
Required girder span (support column spacing)	=	14 ft
Joist span (both sides of girder)	=	12 ft
Species	=	Southern Pine, No. 1
Maximum girder depth	=	12

Find Minimum number of 2x10s or 2x12s required for the built-up girder.

Solution

1. Calculate the design load

W = (Trib. floor joist span)(D + L) = (12 ft)(40 psf + 10 psf) = 600 plf

2. Determine tabulated design values (NDS-S Table 4B)

F_b = 1250 psi $F_{C\perp}$ = 565 psi
F_v = 90 psi E = $1.7\text{x}10^6$ psi

3. Lumber property adjustments (Section 5.2.4):

C_r = 1.2 (Table 5.4) C_D = 1.0
C_F = 1.0 C_b = 1.0
C_H = 2.0 C_L = 1.0

(compression flange laterally braced by connection of floor joists to top or side of girder)

$$F_b' = F_bC_DC_rC_FC_L = 1{,}250 \text{ psi } (1.0)(1.2)(1)(1) = 1{,}500 \text{ psi}$$
$$F_V' = F_lC_DC_H = 90 \text{ psi } (1.25)(2.0) = 225 \text{ psi}$$
$$F_{c\perp}' = F_{c\perp}C_b = 565 \text{ psi } (1) = 565 \text{ psi}$$
$$E' = E = 1.7\text{x}10^6 \text{ psi}$$

4. Determine number of members required due to bending

$$M_{max} = \frac{w\ell^2}{8} = \frac{(600\text{plf})(14\text{ ft})^2}{8} = 14{,}700 \text{ ft-lb}$$
$$f_b = \frac{M}{S} = \frac{(14{,}700 \text{ ft}-\text{lb})(12 \text{ in/ft})}{S} = \frac{176{,}400}{S}$$
$$f_b \le F_b'$$
$$\frac{176{,}400}{S} \le 1{,}500 \text{ psi}$$
$$S_x = 118 \text{ in}^3$$

Using Table 1B in NDS-S

5 2x10s S = 5(21.39) = 107 < 118 (marginal, but 5 too thick)
4 2x12s S = 4 (31.64) = 127 > 118 (OK)

5. Determine number of members required due to horizontal shear

$$V_{max} = \frac{w\ell}{2} = \frac{600\ \text{plf}\ (14\ \text{ft})}{2} = 4{,}200\ \text{lb}$$

$$f_v = \frac{3V}{2A} = \frac{3}{2}\left(\frac{4200}{A}\right) = 6{,}300\ \text{lb}/A$$

$$f_v \leq F_v'$$

$$\frac{6{,}300\ \text{lb}}{A} \leq 225\ \text{psi}$$

$$A = 28\ \text{in}^2$$

2 2x12s A = 33.8 > 28 OK

2 2x10s A = 27.8 ≈ 28 OK

6. Determine required bearing length using 4 2x12s

$$R_1 = R_2 = V_{max} = 4{,}200\ \text{lb}$$

$$f_{c\perp} = \frac{R}{A_b} = \frac{4{,}200\ \text{lb}}{(6\ \text{in})(\ell_b)} = \frac{700}{\ell_b}$$

$$f_{c1} \leq F_{c\perp}'$$

$$\frac{700}{\ell_b} \leq 565\ \text{psi}$$

$$\ell_b = 1.24\ \text{in} \quad \text{(OK)}$$

7. Determine member size due to deflection

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(480\ \text{plf})^*(14\ \text{ft})^4\ (1{,}728\ \text{in}^3/\text{ft}^3)}{384\,EI} = \frac{4.15\times10^8}{EI}$$

*includes 40 psf live load only

$$\rho_{all} \leq \ell/360 = 14\ \text{ft}\,(12\ \text{in/ft})/360 = 0.47\ \text{in}$$

$$\rho_{max} \leq \rho_{all}$$

$$\frac{4.15\times10^8}{EI} = 0.47\ \text{in}$$

$$EI = 8.8\times10^8$$

$$(1.7\times10^6)(I) = 8.8\times10^8$$

$$I = 519\ \text{in}^4$$

3 2x12s I = 534 > 519 okay

8. Check girder for floor system vibration control (see Section 5.3.2)

Girder span, $\ell_1 = 14$ ft
Joist span, $\ell_2 = 12$ ft
$\ell_{TOTAL} = 26$ ft > 20 ft

Therefore, check girder using $\ell/480$ or $\ell/600$ to stiffen floor system

Try $\ell/480$

$$\rho_{max} = \frac{4.15 \times 10^8}{EI} \text{ (as before)}$$

$$\rho_{all} = \ell/480 = \frac{14 \text{ ft}(12 \text{ in/ft})}{480} = 0.35 \text{ in}$$

$$\rho_{max} \leq \rho_{all}$$

$$\frac{4.15 \times 10^8}{EI} = 0.35 \text{ in}$$

$$EI = 1.2 \times 10^9$$

$$I = \frac{1.2 \times 10^9}{1.7 \times 10^6} = 706 \text{ in}^4$$

Using Table 1B in NDS, use

4 2x12s $I = 4\,(178 \text{ in}^4) = 712 \text{ in}^4 > 706 \text{ in}^4$ OK

Conclusion

The bending stress limits the floor girder design to 4 2x12's (No. 1, SYP). The use of 4 2x12s also provides a "stiff" girder with respect to floor vibration (i.e., deflection limit of $\ell/480$. As a practical alternative, a steel "floor beam" (e.g., W-shape) or an engineered wood beam may also be used, particularly if "clearance" is a concern.

EXAMPLE 5.5 *Subfloor Sheathing Design*

Given

Joist spacing = 16 in on center
Floor live load = 40 psf
Use APA rated subflooring

Find The required sheathing span rating and thickness with the face grain perpendicular to the joist span.

Determine size and spacing of fasteners.

Solution

Determine sheathing grade and span rating and thickness by using the APA's *Design and Construction Guide for Residential and Commercial* (APA, 1998). From Table 7 in the APA guide, use 7/16-inch-thick (24/16 rating) sheathing or 15/32-inch- to 1/2-inch-thick (32/16 rating) sheathing. The first number in the rating applies to the maximum spacing of framing members for roof applications; the second to floor applications. It is fairly common to up size the sheathing to the next thickness, e.g., 3/4-inch, to provide a stiffer floor surface. Such a decision often depends on the type of floor finish to be used or the desired "feel" of the floor. Similar ratings are also available from other structural panel trademarking organizations and also comply with the PS-2 standard. It is important to ensure that the sheathing is installed with the long dimension (i.e., face grain) perpendicular to the floor framing; otherwise, the rating does not apply. For wall applications, panel orientation is not an issue.

Use 6d common nails for 7/16-inch-thick sheathing or 8d common nails for thicknesses up to 1 inch (see Table 5.7). Nails should be spaced at 6 inches on center along supported panel edges and 12 inches on center along intermediate supports.

Conclusion

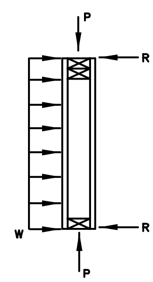
Sheathing design involves matching the proper sheathing rating with the floor framing spacing and live load condition. The process is generally a "cook book" method that follows tables in manufacturer's literature or the applicable building code. Board sheathing and decking are other possible subfloor options that may be designed by using the NDS. Prescriptive tables for these options are also generally available in wood industry publications or in the applicable residential building code.

EXAMPLE 5.6 *Exterior Bearing Wall Design*

Given

Stud size and spacing	=	2x4 at 24 in on center
Wall height	=	8 ft
Species and grade	=	Spruce-Pine-Fir, Stud Grade
Exterior surface	=	7/16-in-thick OSB
Interior surface	=	1/2-in-thick, gypsum wall board
Wind load (100 mph, gust)	=	16 psf (see Chapter 3, Example 3.2)

Find Vertical load capacity of stud wall system for bending (wind) and axial compression (dead load) and for axial compression only (i.e., dead, live, and snow loads); refer to Chapter 3, Table 3.1, for applicable load combinations.



Wall Loading Diagram

Solution

1. Determine tabulated design values for the stud by using the NDS-S (Table A4)

F_b = 675 psi $\quad$ $F_{c\perp}$ = 425 psi

F_t = 350 psi $\quad$ F_c = 725 psi

F_v = 70 psi $\quad$ E = $1.2\text{x}10^6$ psi

2. Determine lumber property adjustments (see Section 5.2.4)

C_D = 1.6 (wind load combination)

= 1.25 (gravity/snow load combination)

C_r = 1.5 (sheathed wall assembly, Table 5.4)

C_L = 1.0 (continuous lateral bracing)

C_F = 1.05 for F_c

= 1.1 for F_t

= 1.1 for F_b

3. Calculate adjusted tensile capacity

Not applicable to this design. Tension capacity is OK by inspection.

4. Calculate adjusted bending capacity

$F_b' = F_bC_DC_LC_FC_r = (675)(1.6)(1.0)(1.1)(1.5) = 1{,}782 \text{ psi}$

5. Calculate adjusted compressive capacity (NDS•3.7)

$F_c^* = F_cC_DC_F = (725 \text{ psi})(1.6)(1.05) = 1{,}218 \text{ psi}$

$E' = E = 1.2\text{x}10^6 \text{ psi}$

$K_{cE} = 0.3$ visually graded lumber

$c = 0.8$ sawn lumber

$$F_{cE} = \frac{K_{cE}E'}{\left(l_e/d\right)^2} = \frac{0.3(1.2\text{ x }10^6 \text{ psi})}{\left[8 \text{ ft}(12 \text{ in/ft})/3.5 \text{ in}\right]^2} = 479 \text{ psi}$$

$$C_p = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$ (column stability factor)

$$= \frac{1+\left(479/1{,}218\right)}{2(0.8)} - \sqrt{\left[\frac{1+\left(479/1{,}218\right)}{2(0.8)}\right]^2 - \frac{479/1{,}218}{0.8}} = 0.35$$

$F_c' = F_cC_DC_FC_P = (725 \text{ psi})(1.6)(1.05)(0.35) = 426 \text{ psi}$

Axial load only case

Calculations are same as above except use $C_D = 1.25$
$F_c^* = 952$ psi
$C_p = 0.44$
$F_c' = F_cC_DC_FC_P = 725 \text{ psi }(1.25)(1.05)(0.44) = 419 \text{ psi}$

6. Calculate combined bending and axial compression capacity for wind and gravity load (dead only) by using the combined stress interaction (CSI) equation (NDS•3.9.2):

$$f_b = \frac{M}{S} = \frac{\frac{1}{8}w\ell^2}{S}$$

$$= \frac{\frac{1}{8}(24 \text{ in})(16 \text{ psf})\left[(8 \text{ ft})(12 \text{ in/ft})\right]^2(1 \text{ ft}/12 \text{ in})}{3.06 \text{ in}^3}$$

$= 1{,}004$ psi

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_b}{F_b'\left[1 - f_c/F_{cE1}\right]} \leq 1.0$$ (CSI equation for bending in strong axis of stud only)

$$\left(\frac{f_c}{426}\right)^2 + \frac{1{,}004}{1{,}782\left(1 - f_c/479\right)} = 1.0$$ (solve CSI equation for f_c)

$f_{c,\,max}$ = 163 psi/stud

P = f_cA = (163 psi/stud)(1.5 in)(3.5 in) = 856 lb/stud

$$w = (856\ \text{lb/stud})\left(\frac{1\ \text{stud}}{2\ \text{ft}}\right) = 428\ \text{plf} \quad \text{(uniform dead load at top of wall)}$$

Therefore, the maximum axial (dead) load capacity is 428 plf with the wind load case (i.e., D+W).

7. Determine maximum axial gravity load without bending load

This analysis applies to the D + L+ 0.3(S or L_r) and D + (S or L_r) + 0.3L load combinations (see Table 3.1, Chapter 3).

Using F_c' determined in Step 5 (axial load only case), determine the stud capacity acting as a column with continuous lateral support in the weak-axis buckling direction.

$$F_c \leq F_c'$$

$$\frac{P}{A} \leq 419\ \text{psi}$$

$$P_{max} = (419\ \text{psi})(1.5\ \text{in})(3.5\ \text{in}) = 2{,}200\ \text{lbs/stud}$$

Maximum axial load capacity (without simultaneous bending load) is 2,200 lbs/stud or 1,100 lbs/lf of wall.

8. Check bearing capacity of wall plate

Not a capacity limit state. ($F_{c\perp}$ is based on deformation limit state, not actual bearing capacity.) OK by inspection.

Conclusion

The axial and bending load capacity of the example wall is ample for most residential design conditions. Thus, in most cases, use of the prescriptive stud tables found in residential building codes may save time. Only in very tall walls (i.e., greater than 10 feet) or more heavily loaded walls than typical will a special analysis as shown here be necessary, even in higher-wind conditions. It is likely that the controlling factor will be a serviceability limit state (i.e., wall deflection) rather than strength, as shown in several of the floor design examples. In such cases, the wall system deflection adjustment factors of Table 5.6 should be considered.

Note:

The axial compression capacity determined above is conservative because the actual EI of the wall system is not considered in the determination of C_p for stability. No method is currently available to include system effects in the analysis of C_p; however, a K_e factor of 0.8 may be used as a reasonable assumption to determine the effective buckling length, ℓ_e, which is then used to determine C_p (see NDS•3.7.1).

Testing has demonstrated that sheathed walls like the one in this example can carry ultimate axial loads of more than 5,000 plf (NAHB/RF, 1974; other unpublished data).

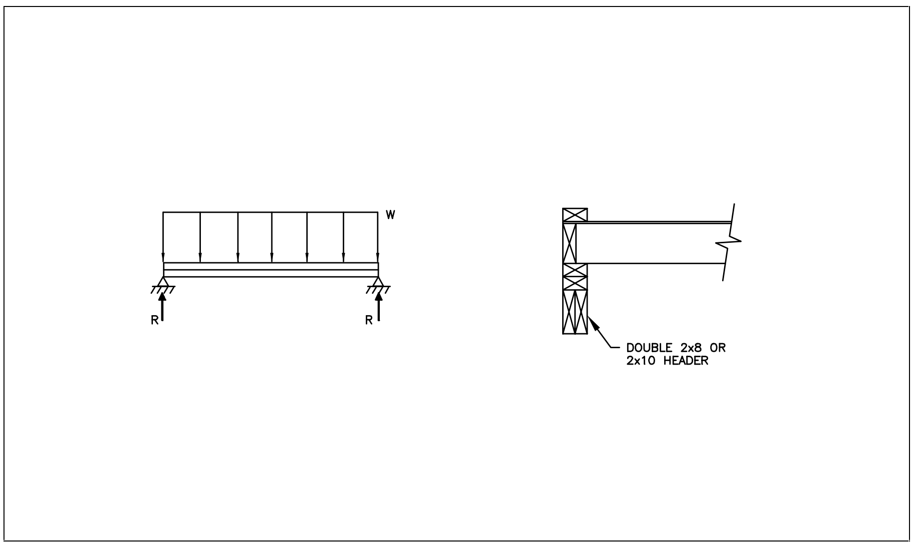
EXAMPLE 5.7 *Header System Design*

Given

Two-story house

Required header span = 6.3 ft (rough opening)
Species and grade = Spruce-Pine-Fir (south), No. 2
Loads on first-story header
w_{floor} = 600 plf (includes floor dead and live loads)
w_{wall} = 360 plf (includes dead, live, and snow loads supported by wall above header)*
w_{total} = 960 plf (includes dead, live, and snow loads)*
*Combined loads are determined in accordance with Table 3.1 of Chapter 3.

Find Determine header size (2x8 or 2x10) by considering system effect of all horizontal members spanning the opening.



Header System

Solution

1. Determine tabulated design values by using the NDS-S (Table 4A)

F_b = 775 psi
F_v = 70 psi
$F_{c\perp}$ = 335 psi
E = 1.1×10^6 psi

2. Determine lumber property adjustments (Section 5.2.4)

C_r = 1.3 (2x10 double header per Table 5.8)
= 1.2 (2x8 double header per Table 5.4)
C_D = 1.25 (snow load)
C_F = 1.1 (2x10)
= 1.2 (2x8)
C_H = 2.0
C_b = 1.0
C_L = 1.0 laterally supported

$F_b' = F_bC_DC_rC_FC_L = (775 \text{ psi})(1.25)(1.3)(1.1)(1.0) = 1{,}385 \text{ psi [2x10]}$
$= (775 \text{ psi})(1.25)(1.2)(1.1)(1.0) = 1{,}279 \text{ psi [2x8]}$
$F_v' = F_vC_DC_H = (70 \text{ psi})(1.25)(2) = 175 \text{ psi}$
$F_{c\perp}' = F_{c\perp}C_b = (335\text{psi})(1) = 335 \text{ psi}$
$E' = E = 1.1\text{x}10^6 \text{ psi}$

With double top plate, F_b can be increased by 5 percent (Table 5.8)

$F_b' = F_b'(1.05) = 1{,}385 \text{ psi } (1.05) = 1{,}454 \text{ psi [2x10]}$
$F_b' = F_b'(1.05) = 1{,}279 \text{ psi } (1.05) = 1{,}343 \text{ psi [2x8]}$

3. Determine header size due to bending for floor load only

$$M_{max} = \frac{w\ell^2}{8} = \frac{(600 \text{ plf})(6.5 \text{ ft})^2}{8} = 3{,}169 \text{ ft-lb}$$
$$f_b = \frac{M_{max}}{S} \leq F_b'$$
$$1{,}454 \text{ psi} = \frac{3{,}169 \text{ ft} - \text{lb}(12 \text{ in/ft})}{S}$$
$$S = 26.2 \text{ in}^3$$
$$S \text{ for 2 2x10} = 2(21.39 \text{ in}^3) = 42.78 \text{ in}^3 > 26.2 \text{ in}^3 \quad \text{(OK)}$$

Try 2 2x8s

$$1{,}343 \text{ psi} = \frac{3{,}169 \text{ ft} - \text{lb}(12 \text{ in/ft})}{S}$$
$$S = 28.3 \text{ in}^3$$
$$S \text{ for 2 2x8} = 2(13.14) = 26.3 \text{ in}^3 < 28.3 \text{ in}^3 \quad \text{(close, but no good)}$$

4. Determine member size due to bending for combined floor and supported wall loads by using the 1.8 system factor from Table 5.8, but not explicitly calculating the load sharing with the band joist above.

$$F_b' = F_b(C_D)(C_r)(C_F)(C_L) = 775 \text{ psi } (1.25)(1.8)(1.1)(1.0) = 1{,}918 \text{ psi}$$
$$M_{max} = \frac{w\ell^2}{8} = \frac{(360 \text{ plf} + 600 \text{ plf})(6.5 \text{ ft})^2}{8} = 5{,}070 \text{ ft-lb}$$
$$f_b = M/S \leq F_b'$$
$$1{,}918 \text{ psi} = \frac{5{,}070 \text{ ft} - \text{lb } (12 \text{ in/ft})}{S}$$
$$S = 31.7 \text{ in}^3$$
$$S \text{ for 2-2x10} = 42.78 \text{ in}^3 > 31.7 \text{ in}^3 \quad \text{(OK)}$$

5. Check horizontal shear

$$V_{max} = \frac{w\ell}{2} = \frac{(600 \text{ plf})(6.5)}{2} = 1{,}950 \text{ lb}$$
$$f_v = \frac{3V}{2A} = \frac{3(1{,}950 \text{ lb})}{2(2)(1.5 \text{ in})(9.25 \text{ in})} = 106 \text{ psi}$$
$$f_v \leq F_v'$$

106 psi < 175 psi (OK)

6. Check for adequate bearing

$$R_1 = R_2 = V_{max} = 1{,}950 \text{ lb}$$

$$f_{c\perp} = \frac{R}{A_b} = \frac{1{,}950 \text{ lb}}{(2)(1.5 \text{ in})(\ell_b)} = \frac{650}{\ell_b}$$

$$f_{c\perp} \leq F_{c\perp}'$$

$$\frac{650}{\ell_b} = 335$$

$\ell_b = 1.9$ in OK for bearing, use 2-2x4 jack studs ($\ell_b = 3$ in)

7. Check deflection

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(600 \text{ plf})(6.5 \text{ ft})^4 (1{,}728 \text{ in}^3/\text{ft}^3)}{384(1.1\text{x}10^6 \text{ psi})[(98.9 \text{ in}^4)(2)]} = 0.11 \text{ in}$$

$$\rho_{all} = L/240 \quad \frac{(6.5 \text{ ft})(12 \text{ in/ft})}{240} = 0.325 \text{ in}$$

$$\rho_{max} < \rho_{all}$$

Conclusion

Using a system-based header design approach, a 2-2x10 header of No. 2 Spruce-Pine-Fir is found to be adequate for the 6 ft-3 in span opening. The loading condition is common to the first story of a typical two-story residential building. Using a stronger species or grade of lumber would allow the use of a 2-2x8 header. Depending on the application and potential savings, it may be more cost-effective to use the header tables found in a typical residential building code. For cost-effective ideas and concepts that allow for reduced header loads and sizes, refer to *Cost Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994). The document also contains convenient header span tables. For headers that are not part of a floor-band joist system, the design approach of this example is still relevant and similar to that used for floor girders. However, the 1.8 system factor used here would not apply, and the double top plate factor would apply only as appropriate.

EXAMPLE 5.8 *Column Design*

Given

Basement column supporting a floor girder
Spruce-Pine-Fir, No. 2 Grade
Axial design load is 4,800 lbs (D + L)
Column height is 7.3 ft (unsupported)

Find Adequacy of a 4x4 solid column

Solution

1. Determine tabulated design values by using the NDS-S (Table 4A)

F_c = 1,150 psi
E = $1.4\text{x}10^6$ psi

2. Lumber property adjustments (Section 5.2.4):

C_D = 1.0
C_F = 1.15 for F_c

3. Calculate adjusted compressive capacity (NDS•3.7):

Trial 4x4

$F_c^* = F_c C_D C_F = 1{,}150 \text{ psi } (1.0)(1.15) = 1{,}323 \text{ psi}$
$E' = E = 1.4\text{x}10^6 \text{ psi}$
K_{cE} = 0.3 for visually graded
c = 0.8 for sawn lumber

$$F_{cE} = \frac{K_{cE}E'}{\left(\ell_e/d\right)^2} = \frac{0.3(1.4\,\text{x}\,10^6\text{psi})}{\left(7.3\text{ ft}(12\text{ in/ft})/3.5\text{ in}\right)^2} = 670 \text{ psi}$$

$$C_p = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

$$= \frac{1+\left(670/1{,}323\right)}{2(0.8)} - \sqrt{\left[\frac{1+\left(670/1{,}323\right)}{2(0.8)}\right]^2 - \frac{670/1{,}323}{0.8}} = 0.44$$

$F_c' = F_c C_D C_F C_p$ = (1,150 psi)(1.0)(1.15)(0.44) = 582 psi
$P_{all} = F'_c A$ = (582 psi)(3.5 in)(3.5 in) = 7,129 lb > 4,800 lb
OK

Conclusion

A 4x4 column is adequate for the 4,800-pound axial design load and the stated height and support conditions. In fact, a greater column spacing could be used. Note that the analysis was performed with a solid sawn column of rectangular dimension. If a nonrectangular column is used, buckling must be analyzed in the weak-axis direction in consideration of the distance between lateral supports, if any, in that direction. If a built-up column is used, it is NOT treated the same way as a solid column. Even if the dimensions are nearly the same, the built-up column is more susceptible to buckling due to slippage between adjacent members as flexure occurs in response to buckling (only if unbraced in the weak-axis direction of the built-up members). Slippage depends on how well the built-up members are fastened together, which is accounted for by the use of an additional adjustment (reduction) factor applied to the C_p equation (see Section 5.5.5 and NDS•15.3).

EXAMPLE 5.9 *Simply Supported Sloped Rafter Design*

Given

Two-story home
Rafter spacing 16 in on center
Rafter horizontal span is 12 ft (actual sloped span is 14.4 ft)
8:12 roof slope
Design loads (see Chapter 3):

Dead load	=	10 psf
Roof snow load	=	20 psf (20 psf ground snow)
Wind load (90 mph, gust)	=	12.7 psf (outward, uplift)
	=	7.4 psf (inward)
Roof live load	=	10 psf

Find Minimum rafter size using No. 2 Douglas-Fir-Larch (refer to Figure 5.7 for load diagram).

Solution

1. Evaluate load combinations applicable to rafter design (see Chapter 3, Table 3.1):

The load combinations to consider and initial assessment based on the magnitude of the given design loads follows

$D + (L_r \text{ or } S)$ Controls rafter design in inward-bending direction (compression side of rafter laterally supported); L_r can be ignored since the snow load magnitude is greater.

$0.6D + W_u$ May control rafter design in outward-bending direction since the compression side now has no lateral bracing unless specified; also important to rafter connections at the bearing wall and ridge beam.

$D + W$ Not controlling by inspection; gravity load D + S controls in the inward-bending direction.

2. Determine relevant lumber property values (NDS-S, Table 4A).

F_b = 900 psi
F_v = 95 psi
E = 1.6×10^6 psi

3. Determine relevant adjustments to property values assuming a 2x8 will be used (Section 5.2.4):

C_D = 1.6 (wind load combinations)
= 1.25 (snow load combination)
C_r = 1.15 (2x8, 24 inches on center)
C_H = 2.0
C_F = 1.2 (2x8)
C_L = 1.0 (inward bending, D + S, laterally braced on compression edge)
= 0.32 (outward bending, 0.6 D + W, laterally unbraced on compression edge)*

*Determined in accordance with NDS•3.3.3

$$\ell_e = 1.63\,\ell_u + 3d$$
$$= 1.63\,(14.4\text{ ft}) + 3\,(7.25\text{ in})(1\text{ in}/12\text{ft})$$
$$= 25.3\text{ ft}$$
$$R_B = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{(25.5\text{ ft})(12\text{ in}/\text{ft})(7.25\text{ in})}{(1.5\text{ in})^2}}$$
$$= 31 \quad < \quad 50\text{ (OK)}$$
$$K_{bE} = 0.439 \quad \text{(visually graded lumber)}$$
$$F_{bE} = \frac{K_{bE}E'}{R_B{}^2} = \frac{0.439\,(1.6\text{x}10^6\text{ psi})}{(31)^2} = 730\text{ psi}$$
$$F_b{*} = F_bC_DC_rC_F$$
$$= 900\text{ psi }(1.6)(1.15)(1.2) = 1{,}987\text{ psi}$$
$$C_L = \frac{1+(F_{bE}/F_b{*})}{1.9} - \sqrt{\left[\frac{1+(F_{bE}/F_b{*})}{1.9}\right]^2 - \frac{F_{bE}/F_b{*}}{0.95}}$$
$$C_L = 0.36 \quad (2\text{x}8)$$

4. Determine rafter transverse bending load, shear, and moment for the wind uplift load case (using Method A of Figure 5.8).

The wind load acts transverse (i.e., perpendicular) to the rafter; however, the snow load acts in the direction of gravity and must be resolved to its transverse component. Generally, the axial component of the gravity load along the rafter (which varies unknowingly depending on end connectivity) is ignored and has negligible impact considering the roof system effects that are also ignored. Also, given the limited overhang length, this too will have a negligible impact on the design of the rafter itself. Thus, the rafter can be reasonably analyzed as a sloped, simply supported bending member. In analyzing wind uplift connection forces at the outside bearing of the rafter, the designer should consider the additional uplift created by the small overhang, though for the stated condition it would amount only to about 20 pounds additional uplift load.

The net uniform uplift load perpendicular to the rafter is determined as follows:

$$w_{D,\text{ transverse}} = w_D\,(\cos\theta)$$
$$= (10\text{ psf})(1.33\text{ ft})(\cos 33.7°)$$
$$= 11\text{ plf}$$
$$w_{w,\text{ transverse}} = (12.7\text{ psf})(1.33\text{ ft}) = 17\text{ plf (uplift)}$$
$$w_{\text{total, transverse}} = 17\text{ plf}-11\text{ plf} = 6\text{ plf (net uplift)}$$
$$\text{Shear, }V_{max} = \frac{w\ell}{2} = \frac{(6\text{ plf})(14.4\text{ ft})}{2} = 44\text{ lbs}$$
$$\text{Moment, }M_{max} = 1/8\,w\ell^2$$
$$= 1/8\,(6\text{ plf})(14.4\text{ ft})^2 = 156\text{ ft-lb}$$

5. Determine bending load, shear, and moment for the gravity load case (D + S) using Method B of Figure 5.8 (horizontal span):

w_D = (10 psf)(14.4 ft)(1.33 ft)/12 ft-horizontal = 16 plf
w_S = (20 psf)(12 ft)(1.33 ft)/12 ft-horizontal = 27 plf
w_{total} = 43 plf
w_{total} = (43 plf)(cos 33.7°) = 36 plf

$$\text{Shear, } V_{max} = \frac{(36\ \text{plf})(12\ \text{ft})}{2} = 216\ \text{lb}$$

$$\text{Moment, } M_{max} = 1/8\ (36\ \text{plf})(12\ \text{ft})^2 = 648\ \text{ft-lb}$$

6. Check bending stress for both loading cases and bending conditions

Outward Bending (0.6D + W_u)

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

$$= \frac{156\ \text{ft}-\text{lb}}{13.14\ \text{in}^3}(12\ \text{in/ft}) = 142\ \text{psi}$$

F_b’ = $F_bC_DC_rC_FC_L$
= 900 psi (1.6)(1.15)(1.2)(0.36) = 715 psi
f_b << F_b’ OK, 2x8 works and no lateral bracing of bottom compression edge is required

Inward Bending (D + S)

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

$$= \frac{648\ \text{ft}-\text{lb}}{13.14\ \text{in}^3}(12\ \text{in/ft}) = 591\ \text{psi}$$

F_b’ = $F_bC_DC_rC_FC_L$
= 900 psi (1.25)(1.15)(1.2)(1.0) = 1,553 psi
f_b << F_b’ (OK)

7. Check horizontal shear

V_{max} = 216 lb (see Step 5)

$$f_v = \frac{3V}{2A} = \frac{3(216\ \text{lb})}{2(1.5\ \text{in})(7.25\ \text{in})} = 30\ \text{psi}$$

F_v’ = $F_vC_DC_H$ = 95 psi (1.25)(2.0) = 238 psi
f_v << F_v’ (OK)

8. Check bearing

OK by inspection.

9. Check deflection criteria for gravity load condition (Section 5.2.2)

$$\rho_{all} = \frac{\ell}{180} = \frac{(14.4\ \text{ft})(12\ \text{in/ft})}{180} = 1.0\ \text{in}$$

$$\rho_{max} = \frac{5w\ell^4}{384\,EI} = \frac{5(36\ \text{plf})(14.4\ \text{ft})^4}{384(1.6 \times 10^6\ \text{psi})(47.6\ \text{in}^4)}\quad (1{,}728\ \text{in}^3/\text{ft}^3)$$

$$= 0.4\ \text{in}$$

$\rho_{max} << \rho_{all}$ (OK, usually not a mandatory roof check)

Conclusion

A 2x8, No. 2 Douglas-Fir-Larch rafter spaced at 16 inches on center was shown to have ample capacity and stiffness for the given design conditions. In fact, using a 19.2 inch on center spacing (i.e., five joists per every 8 feet) would also work with a more efficient use of lumber. It is also possible that a 2x6 could result in a reasonable rafter design for this application. For other concepts in value-added framing design, consult *Cost Effective Home Building: A Design and Construction Handbook* (NAHBRC, 1994). The document also contains prescriptive span tables for roof framing design.

EXAMPLE 5.10 *Ridge Beam Design*

Given

One-story building
Ridge beam span = 13 ft
Roof slope = 6:12
Rafter horizontal span = 12 ft

Loading (Chapter 3)

Dead	=	15 psf
Snow	=	20 psf
Wind (110 mph, gust)	=	6.3 psf (inward)
	=	14.2 psf (outward, uplift)
Live	=	10 psf

Find Optimum size and grade of lumber to use for a solid (single-member) ridge beam.

Solution

1. Evaluate load combinations applicable to the ridge beam design (see Chapter 3, Table 3.1)

$D + (L_r \text{ or } S)$ Controls ridge beam design in the inward-bending direction (compression side of beam laterally supported by top bearing rafters); L_r can be ignored because the roof snow load is greater.

$0.6\ D + W_u$ May control ridge beam design in outward-bending direction because the bottom (compression side) is laterally unsupported (i.e., exposed ridge beam for cathedral ceiling); also important to ridge beam connection to supporting columns. However, a ridge beam supporting rafters that are tied-down to resist wind uplift cannot experience significant uplift without significant upward movement of the rafters at the wall connection, and deformation of the entire sloped roof diaphragm (depending on roof slope).

$D + W$ Not controlling because snow load is greater in the inward direction; also, positive pressure is possible only on the sloped windward roof surface while the leeward roof surface is always under negative (suction) pressure for wind perpendicular to the ridge; the case of wind parallel to the ridge results in uplift across both sides of the roof, which is addressed in the $0.6\ D + W_u$ load combination and the roof uplift coefficients in Chapter 3 and based on this worst case wind direction.

2. Determine the ridge beam bending load, shear, and moment for the wind uplift load case

In accordance with a procedure similar to Step 4 of Example 5.9, the following ridge beam loads are determined:

Rafter sloped span = horizontal span/cos θ
= 12 ft/cos 26.6°
= 13.4 ft

Load on ridge beam

w_{dead} = (rafter sloped span)(15 psf)
[1/2 rafter span on each side]
= (13.4 ft)(15 psf)
= 201 plf

0.6 w_{dead} = 121 plf

w_{wind} = (13.4 ft)(14.2 psf) cos 26.6°
= 170 plf

w_{total} = 170 plf - 121 plf = 49 plf (outward or upward)

Shear, V_{max} = 1/2 $w\ell$ = 1/2 (49 plf)(13 ft)
= 319 lb

Moment, M_{max} = 1/8 $w\ell^2$ = 1/8 (49 plf)$(13\text{ft})^2$
= 1,035 ft-lb

Note: If the rafters are adequately tied-down to resist uplift from wind, the ridge beam cannot deform upward without deforming the entire sloped roof diaphragm and the rafter-to-wall connections. Therefore, the above loads should be considered with reasonable judgment. It is more important, however, to ensure that the structure is appropriately tied together to act as a unit.

3. Determine the ridge beam loading, shear, and moment for the D + S gravity load case

D + S = 15 psf + 20 psf = 35 psf
(pressures are additive because both are gravity loads)
load on ridge beam

W_{D+S} = (13.4 ft)(35 psf) = 469 plf
Shear, V_{max} = 1/2 (469 plf)(13 ft) = 3,049 lb
Moment, M_{max} = 1/8 (469 plf)$(13\text{ ft})^2$ = 9,908 ft-lb

4. Determine the optimum ridge beam size and grade based on the above bending loads and lateral support conditions.

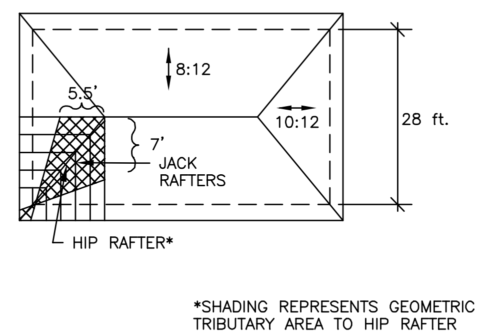
Note. The remainder of the problem is essentially identical to Example 5.9 with respect to determining the strength of the wood member. However, a trial member size and grade are needed to determine the lumber stresses as well as the lumber property adjustment values. Thus, the process of optimizing a lumber species, size, and grade selection from the multitude of choices is iterative and time consuming by hand calculation. Several computerized wood design products on the market can perform the task. However, the computerized design procedures may not allow for flexibility in design approach or assumptions if the designer is attempting to use recommendations similar to those given in this guide. For this reason, many designers prefer to create their own analysis spreadsheets as a customized personal design aid. The remainder of this problem is left to the reader for experimentation.

EXAMPLE 5.11 *Hip Rafter Design*

Given

One-story building
Hip rafter and roof plan as shown below
Rafters are 2x8 No. 2 Hem-Fir at 16 in on center
Loading (see Chapter 3)

Dead	=	10 psf
Snow	=	10 psf
Wind (90 mph, gust)	=	4 psf (inward)
	=	10 psf (uplift)
Live (roof)	=	15 psf



Roof Plan, Hip Rafter Framing, and Tributary Load Area

Find

1. Hip rafter design approach for rafter-ceiling joist roof framing.
2. Hip rafter design approach for cathedral ceiling framing (no cross-ties; ridge beam and hip rafter supported by end-bearing supports).

Solution

1. Evaluate load combinations applicable to the hip rafter design (see Chapter 3, Table 3.1)

By inspection, the D + L_r load combination governs the design. While the wind uplift is sufficient to create a small upward bending load above the counteracting dead load of 0.6 D, it does not exceed the gravity loading condition in effect. Since the compression edge of the hip rafter is laterally braced in both directions of strong-axis bending (i.e., jack rafters frame into the side and sheathing provides additional support to the top), the 0.6 D + W_u condition can be dismissed by inspection. Likewise, the D + W inward-bending load is considerably smaller than the gravity load condition. However, wind uplift should be considered in the design of the hip rafter connections; refer to Chapter 7.

2. Design the hip rafter for a rafter-ceiling joist roof construction (conventional practice).

Use a double 2x10 No. 2 Hem-fir hip rafter (i.e., hip rafter is one-size larger than rafters - rule of thumb). The double 2x10 may be lap-spliced and braced at or near mid-span; otherwise, a single 2x10 could be used to span continuously. The lap splice should be about 4 feet in length and both members face-nailed together with 2-10d common nails at 16 inches on center. Design is by inspection and common practice.

Note: The standard practice above applies only when the jack rafters are tied to the ceiling joists to resist outward thrust at the wall resulting from truss action of the framing system. The roof sheathing is integral to the structural capacity of the system; therefore, heavy loads on the roof before roof sheathing installation should be avoided, as is common. For lower roof slopes, a structural analysis (see next step) may be warranted because the folded-plate action of the roof sheathing is somewhat diminished at lower slopes. Also, it is important to consider connection of the hip rafter at the ridge. Usually, a standard connection using toe-nails is used, but in high wind or snow load conditions a connector or strapping should be considered.

3. Design the hip rafter by assuming a cathedral ceiling with bearing at the exterior wall corner and at a column at the ridge beam intersection

a. Assume the rafter is simply supported and ignore the negligible effect of loads on the small overhang with respect to rafter design.

b. Simplify the diamond-shaped tributary load area (see figure above) by assuming a roughly "equivalent" uniform rectangular load area as follows:

Tributary width ≈ 4 ft

$w_{D+S} = (10 \text{ psf} + 15 \text{ psf})(4 \text{ ft}) = 100 \text{ plf}$

c. Determine the horizontal span of the hip rafter based on roof geometry:

$$\text{Horizontal hip span} = \sqrt{(14\,\text{ft})^2 + (11\,\text{ft})^2} = 17.8 \text{ ft}$$

d. Based on horizontal span (Method B, Figure 5.8), determine shear and bending moment:

$$\text{Shear, } V_{max} = 1/2\, w\ell = 1/2\,(100 \text{ plf})(17.8 \text{ ft}) = 890 \text{ lb}$$

$$\text{Moment, } M_{max} = 1/8\, w\ell^2 = 1/8\,(100 \text{ plf})(17.8 \text{ ft})^2 = 3{,}960 \text{ ft-lb}$$

f. Determine required section modulus assuming use of 2x12 No. 2 Hem-Fir

$$f_b = \frac{M}{S} = \frac{3{,}960 \text{ ft} - \text{lb}}{S}(12 \text{ in/ft}) = \frac{47{,}520 \text{ in} - \text{lb}}{S}$$

$F_b' = F_b C_D C_r C_F C_L$ (F_b from NDS-S, Table 4A)

$F_b' = 850 \text{ psi } (1.25)(1.0)(1.0)(1.0) = 1{,}063 \text{ psi}$

$f_b \leq F_b'$

$$\frac{47{,}520 \text{ in} - \text{lb}}{S_{REQ'D}} = 1{,}063 \text{ psi}$$

$S_{REQ'D} = 44.7 \text{ in}^3$

$S_{2x12} = 31.6 \text{ in}^3$

Therefore, 2-2x12s are required because of bending.

Try 2-2x10s,

$$F_b' = (850\text{ psi})(1.25)(1.2)(1.1)(1.0) = 1{,}403\text{ psi}$$

$$\frac{47{,}520\text{ in}-\text{lb}}{S_{REQ'D}} = 1{,}403\text{ psi}$$

$$S_{REQ'D} = 34\text{ in}^3$$

$$S_{2x10} = 21.39\text{ in}^3$$

Therefore, 2-2x10s are acceptable (2x21.39 in^3 = 42.8 in^3).

g. Check horizontal shear:

$$f_v = \frac{3V}{2A} = \frac{3\ (890\text{ lb})}{2\ (2)(1.5\text{ in})(9.25\text{ in})} = 48.1\text{ psi}$$

$$f_v \ll F_v'$$

OK by inspection

h. Consider deflection:

Deflection is OK by inspection. No method exists to accurately estimate deflection of a hip rafter that is subject to significant system stiffness because of the folded-plate action of the roof sheathing diaphragm.

Conclusion

Use 2-2x10 (No. 2 Hem-Fir) for the hip rafters for the cathedral ceiling condition (not considering sloped roof sheathing system effects). However, a cathedral ceiling with a hip roof is not a common occurrence. For traditional rafter-ceiling joist roof construction, a hip rafter one or two sizes larger than the rafters can be used, particularly if it is braced at or near mid-span. With a ceiling joist or cross-ties, the ridge member and hip rafter member need only serve as plates or boards that provide a connection interface, not a beam, for the rafters.

5.8 References

AF&PA, *Commentary on the National Design Specification for Wood Construction*, American Forest and Paper Association, Washington, DC, 1999.

AF&PA, *Load and Resistance Factor Design (LRFD) Manual for Engineered Wood Construction*, American Forest and Paper Association, Washington, DC, 1996a.

AF&PA, *National Design Specification for Wood Construction and Supplement*, American Forest and Paper Association, Washington, DC, 1997.

AF&PA, *Wood Frame Construction Manual–SBC High Wind Edition*, American Forest and Paper Association, Washington, DC, 1996b.

AISC, *Manual of Steel Construction Allowable Stress Design, Ninth Edition*, American Institute of Steel Construction, Chicago, IL, 1989.

AISI, *Residential Steel Beam and Column Load/Span Tables*, Publication RG-936, American Iron and Steel Institute, Washington, DC, June 1993.

APA, *Design and Construction Guide: Residential and Commercial*, APA–The Engineered Wood Association, Tacoma, WA, 1998a.

APA, *Load-Span Tables for APA Structural-Use Panels*, APA–The Engineered Wood Association, Tacoma, WA, January 1999.

APA, *Plywood Floors for Residential Garages*, Report 139, APA–The Engineered Wood Association, Tacoma, WA, 1980.

APA, *Product Design Specification (PDS)*, American Plywood Association APA–Engineered Wood Association, Tacoma, WA, 1998b.

ASAE, *Design Requirements and Bending Properties for Mechanically Laminated Columns* (EP 559), American Society of Agricultural Engineers, St. Joseph, MI, 1997.

ASTM, *Standard Practice for Evaluating Allowable Properties for Grades of Structural Lumber* (D2915-94), American Society of Testing and Materials, West Conshohocken, PA, 1997.

ASTM, *Standard Practice for Establishing Allowable Properties for Visually-Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens* (D1990-97), American Society of Testing and Materials, West Conshohocken, PA, 1998a.

ASTM, *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber* (D245-98), American Society of Testing and Materials, West Conshohocken, PA, 1998c.

ASTM, *Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design* (D5457-93), American Society of Testing and Materials, West Conshohocken, PA, 1998b.

Bonnicksen, L.W. and Suddarth, S.K., *Structural Reliability Analysis of Wood Load Sharing Systems*, Paper No. 82, American Society of Testing and Materials, Fifth National Meeting, Philadelphia, Pennsylvania, 1965.

Douglas, B.K. and Line, P., "System Effects in Wood Assemblies," *Proceedings of the International Wood Engineering Conference*, New Orleans, LA, 1996.

FHA, *Minimum Property Standards for One and Two Living Units*, FHA No. 300, Federal Housing Administration, Washington, DC, November 1, 1958.

Forest Products Laboratory, *Wood Engineering Handbook, Second Edition*, Prentice Hall, Englewood Cliffs, NJ, 1990.

FPRS, *Wall & Floor Systems: Design and Performance of Light Frame Structures*, Proceedings 7317, Forest Products Research Society, Madison, WI, 1983.

Galambos, T.V. and Ellingwood, B., "Serviceability Limit States: Deflection," *Journal of Light Engineering*, Vol. 112, No. 1, American Society of Civil Engineers, Reston, VA, January 1986.

Gillespie, R.H., Countryman, D., and Blomquist, R.F., *Adhesives in Building Construction* (Chapter 3: Structural Design Considerations), Agriculture Handbook No. 516, U.S. Department of Agriculture, Washington, DC, 1978.

HUD, *Prevention and Control of Decay in Homes*, prepared by the Southern Forest Experiment Station for the U.S. Department of Housing and Urban Development, Washington, DC, 1978.

HUD, *System Performance of Wood Header Assemblies*, prepared by the NAHB Research Center, Inc., for the U.S. Department of Housing and Urban Development, Washington, DC, 1999.

Hurst, H.T., *The Wood-Frame House as a Structural Unit*, National Forest Products Association, Washington, DC, 1965.

ICC, *International One- and Two-Family Dwelling Code* (formerly the *CABO One- and Two-Family Dwelling Code*), International Code Council, Inc., Falls Church, VA, 1998.

Mtenga, P.V., "Impact of Using a System Factor in the Design of Light-Frame Roof Assemblies," *Forest Products Journal*, Vol. 48 No. 5, 27-34, 1998.

NAHB, *Bridging in Residential Floor Construction*, LR-6, National Association of Home Builders, Washington, DC, 1961.

NAHB, *Cost Effective Home Building: A Design and Construction Handbook*, prepared by the NAHB Research Center, Inc., for the National Association of Home Builders, Home Builder Press, Washington, DC, 1994.

NAHBRC, *NAHB Beam Series*, prepared by the NAHB Research Center, Inc., for the National Association of Home Builders, NAHB, Washington, DC, 1981.

NAHBRC, *Prescriptive Method for Residential Cold-Formed Steel Framing, Third Edition*, NAHB Research Center, Inc., Upper Marlboro, MD, 1998.

NAHBRC, *Truss-Framed Construction*, NAHB Research Center, Inc., Upper Marlboro, MD, 1982.

NAHBRF, *Stress and Deflection Reduction in 2x4 Studs Spaced 24 Inches on Center Due to the Addition of Interior and Exterior Surfacings*, NAHB Research Foundation, Rockville, MD, July 1974.

NES, *Pneumatic or Mechanically Driven Staples, Nails, P-Nails, and Allied Fasteners for Use in All Types of Building Construction*, Report No. NER-272, National Evaluation Service, Inc., Falls Church, VA, 1997.

NIST, *American Softwood Lumber Voluntary Product Standard* (USDOC PS-20) National Institute of Standards and Technology, Gaithersburg, MD, 1994.

NIST, *Voluntary Performance Standard for Wood-Based Structural-Use Panels* (USDOC PS-2), National Institute of Standards and Technology (NIST), Gaithersburg, MD, 1992.

NIST, *Voluntary Product Standard for Construction and Industrial Plywood* (USDOC PS-1), National Institute of Standards and Technology, Gaithersburg, MD, 1995.

Pellicane, P.J. and Anthony, R.W., "Effect of Adhesives on the Deflection Behavior of Light-Frame Wood Floors," *Proceedings of the International Wood Engineering Conference*, Vol. 3, Omnipress, Madison, WI, 1996.

Polensek, A., *Rational Design Procedure for Wood Stud Walls under Bending and Compression Loads*, Forest Research Laboratory, Oregon State University, September 1975.

Rosowsky, D. and Ellingwood, B., "Reliability of Wood Systems Subjected to Stochastic Live Loads", *Wood and Fiber Science*, Society of Wood Science and Technology, Madison, WI, 1992.

SBCCI, *Standard Building Code*, Southern Building Code Congress International, Birmingham, AL, 1999.

TPI, *Commentary and Appendices to the National Design Standard for Metal Plate Connected Wood Truss Construction* (TPI-1), Truss Plate Institute, Madison, WI, 1995b.

TPI, *National Design Standard for Metal Plate Connected Wood Truss Construction* (TPI-1), Truss Plate Institute, Madison, WI, 1995a.

Triche, M.H. and Suddarth, S.K., *Purdue Plane Structures Analyzer 4*, Purdue Research Foundation, West Lafayette, IN, 1993.

Tucker, B.J. and Fridley, K.J., "Concentrated Load Design Procedures for Wood-Joist Floor Systems," *Journal of Structural Engineering*, Vol. 125, No. 7, July 1999.

Woeste, L.F. and Dolan, J.D., "Beyond Code: Preventing Floor Vibration," *Journal of Light Construction*, November 1998, 69-71.

Wolfe, R.W., "Performance of Light-Frame Redundant Assemblies," *Proceedings of 1990 International Timber Engineering Conference*, Vol. 1, 124-131, 1990.

Wolfe, R.W., "Structural Performance of Light-Frame Truss-Roof Assemblies," *Proceedings of the International Wood Engineering Conference*, Vol. 3, Omnipress, Madison, WI, 1996.

Wolfe, R.W. and LaBissoniere, T., *Structural Performance of Light-Frame Roof Assemblies, II. Conventional Truss Assemblies*, FPL-RP-499, USDA Forest Products Laboratory, Madison, WI, 1991.

WTCA, *Metal Plate Connected Wood Truss Handbook*, prepared by E. Callahan for the Wood Truss Council of America, Madison, WI, 1997.

WTCA, *Standard Practice for Metal Plate Connected Wood Truss Design Responsibilities* (WTCA 1-95), Wood Truss Council of America, Madison, WI, 1995.

WTCA, *Commentary for Permanent Bracing of Metal Plate Connected Wood Trusses*, Wood Truss Council of America, Madison, WI, 1999.