CHAPTER 5

Design of Wood Framing

5.1 General

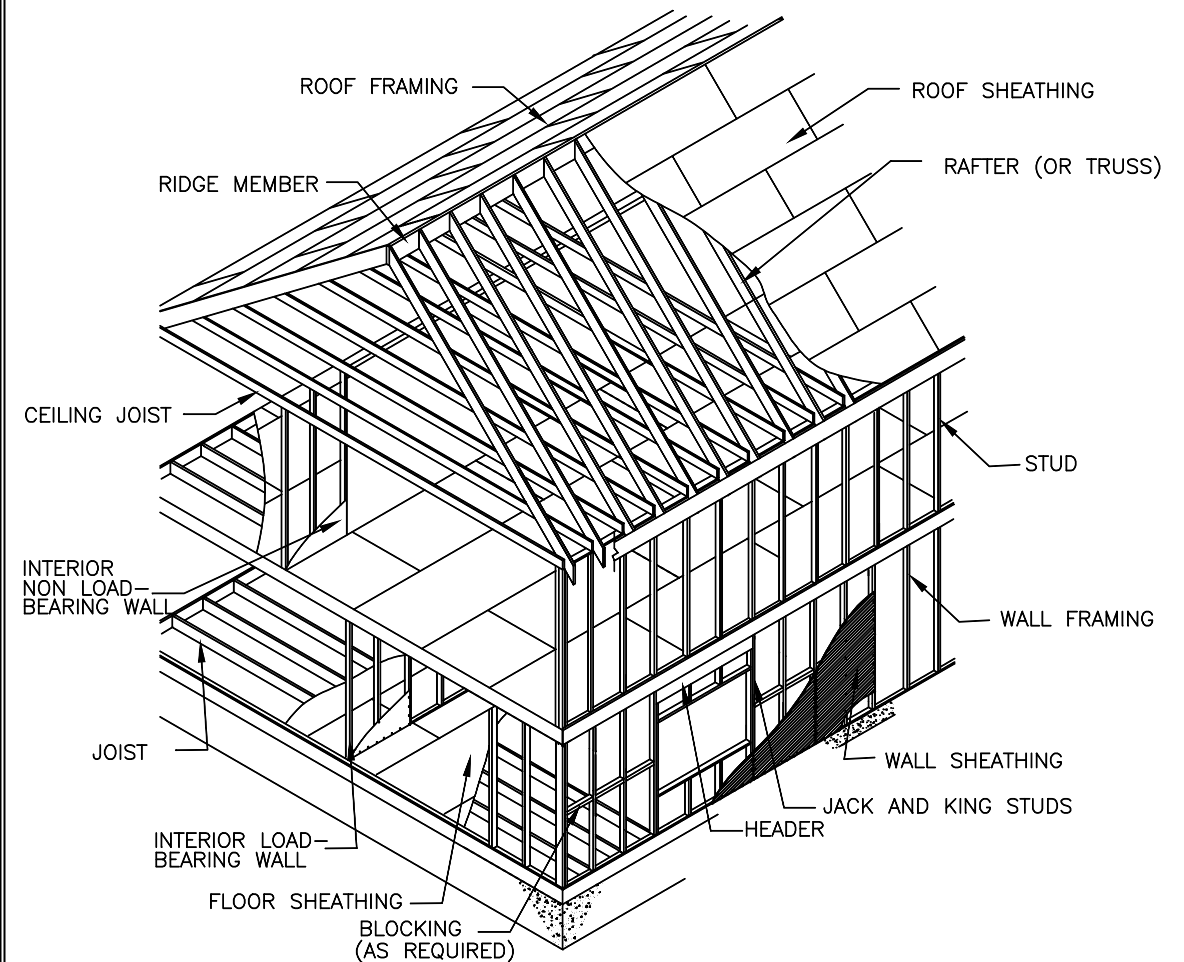
This chapter addresses elements of above-grade structural systems in residential construction. As discussed in Chapter 1, the residential construction material most commonly used above grade in the United States is light-frame wood; therefore, this chapter focuses on structural design that specifies 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]

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

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

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

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

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

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

Horizontal Shear

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

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

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

[NDS•3.4]

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

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

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

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

Compression Perpendicular to Grain (Bearing)

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

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

[NDS•3.10]

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

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

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

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

Combined Bending and Axial Loading

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

[NDS•3.9]

Combined bending and axial tension design check

$$\frac{f_t}{F'_t} + \frac{f_b}{F_b^*} \le 1$$

$$\frac{f_b - f_t}{F_b^{**}} \le 1$$

Combined bending and axial compression design check

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

Compression and Column Stability

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

[NDS•3.7]

$f_c \leq F'_c$ 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.

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

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

$F_c^* = F_c$ 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]

$f_t \leq F'_t$ 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]

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

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

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).

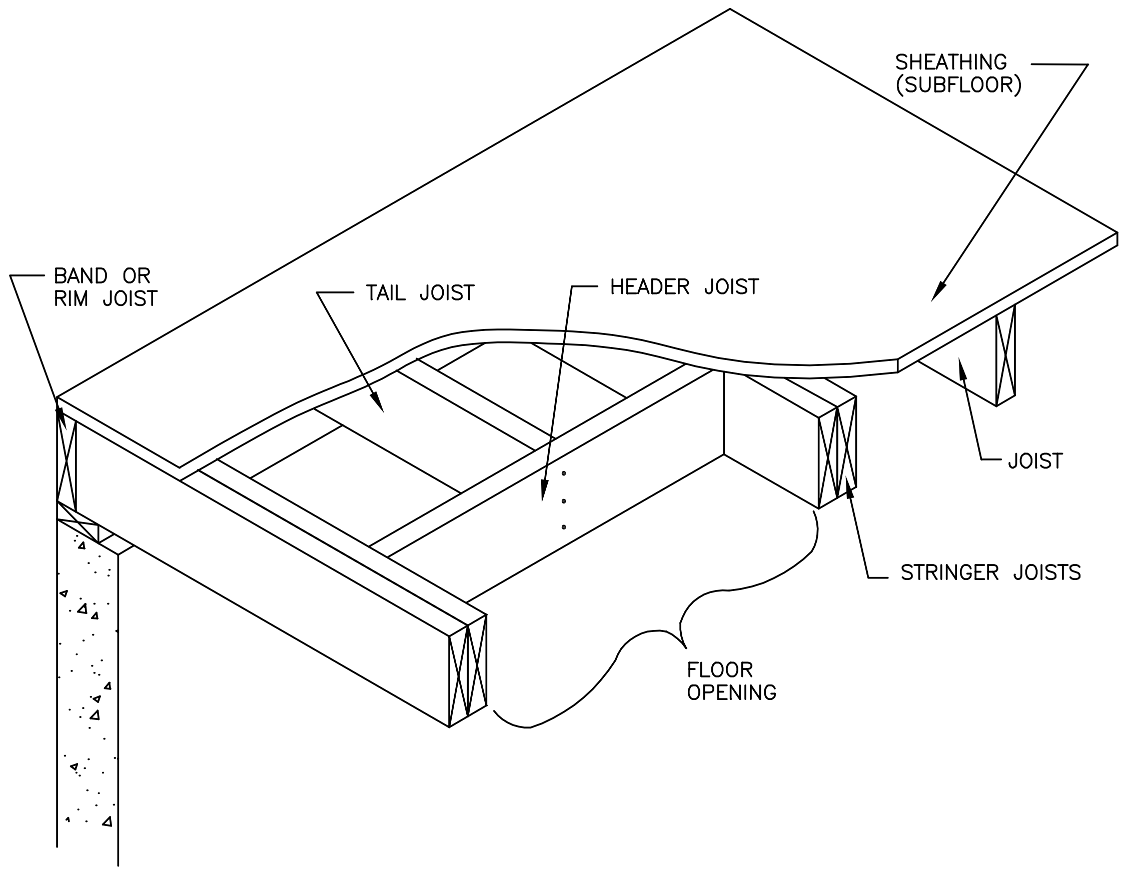
5.4.1 General

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:

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

5.4.2 Floor Joist Design

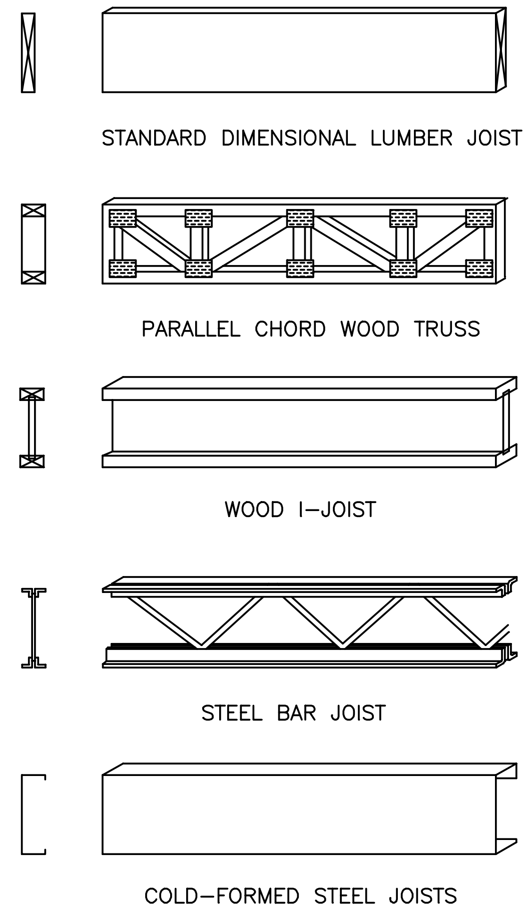
Readily available tables in residential building codes provide maximum allowable spans for different species, grades, sizes, and spacings of lumber joists. Some efficient concepts for floor joist design are also provided in *Cost Effective Home Building: A Design and Construction Handbook* (NAHB, 1994).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

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

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

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

FIGURE 5.3 ***Conventional and Alternative Floor Framing Members***



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

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

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

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

5.4.3 Girder Design

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

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

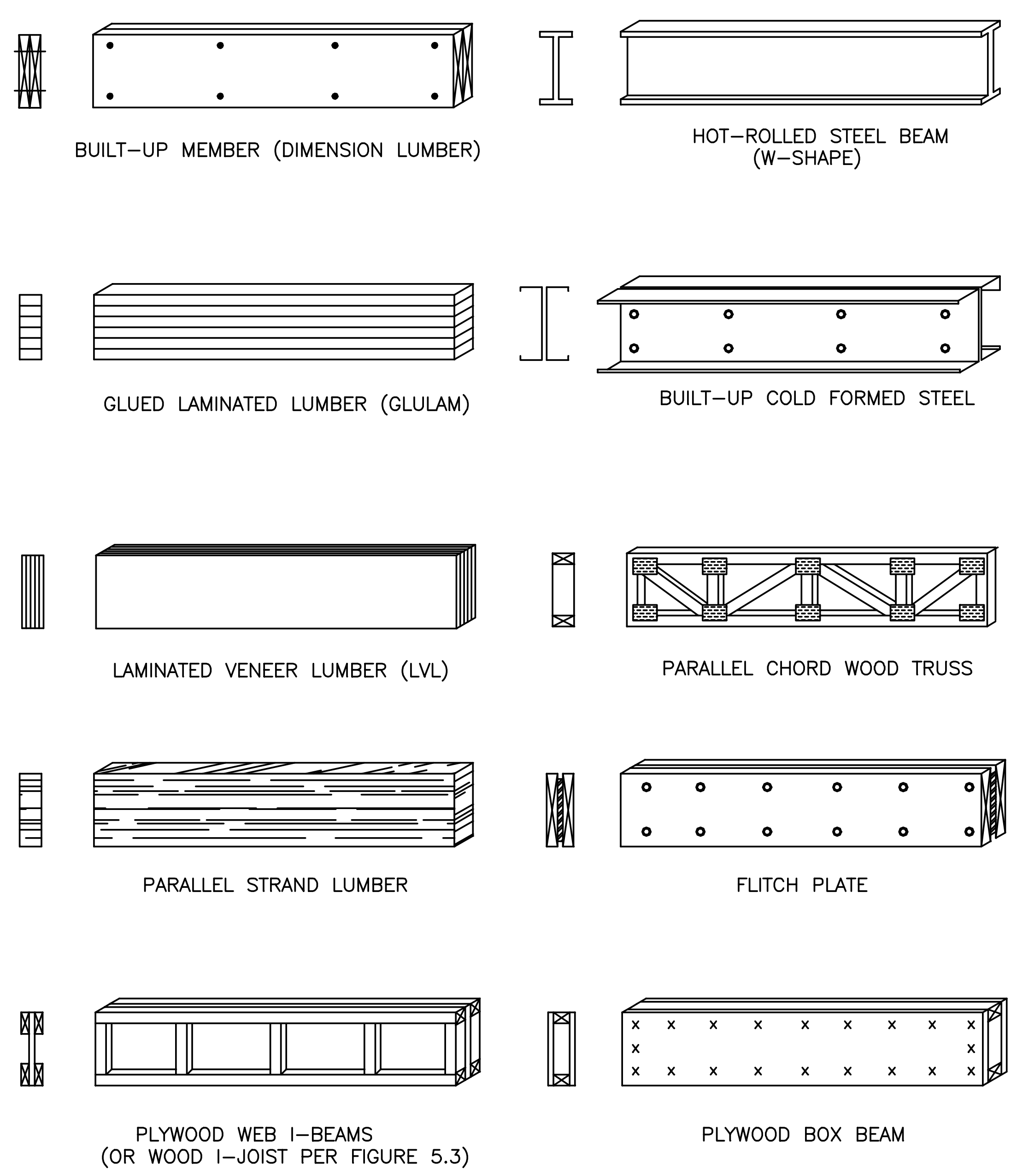
Built-up beams are constructed by nailing together of two or more plys of dimension lumber. Since load sharing occurs between the plys (i.e., lumber members), the built-up girder is able to resist higher loads than a single member of the same overall dimensions. The built-up member can resist higher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

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

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

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

FIGURE 5.4 Examples of Beams and Girders



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

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

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

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

5.4.4 Subfloor Design

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

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

TABLE 5.7 ***Fastening Floor Sheathing to Structural Members***[1]

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

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

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

5.5 Wall Framing

The objectives of wall system design are

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

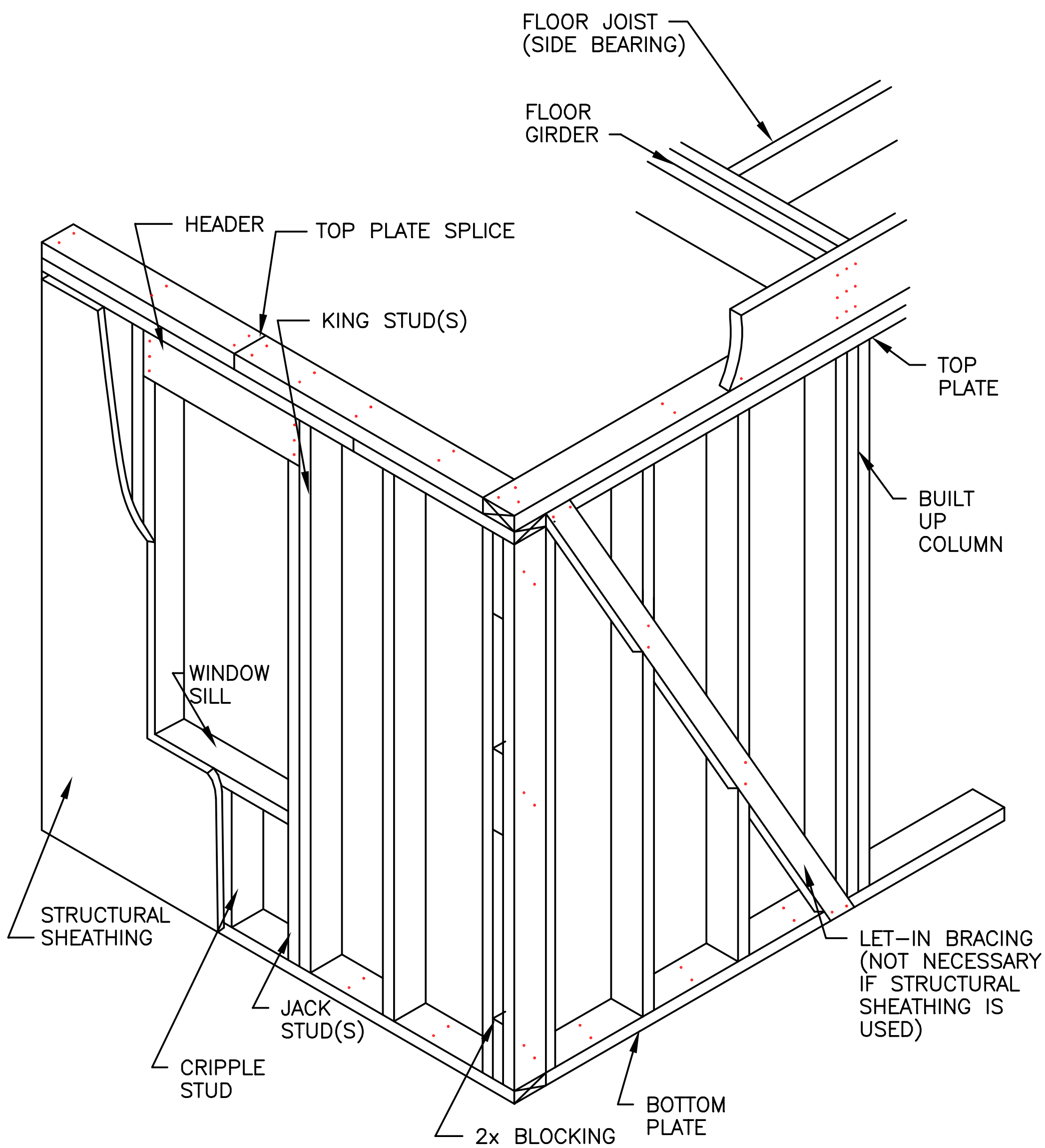
5.5.1 General

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

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

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

FIGURE 5.5 ***Structural Elements of the Wall System***



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

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

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

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

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

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

The third load combination provides a check for the out-of-plane bending condition due to lateral wind loads on the wall. For tall wood-frame walls that support heavy claddings such as brick veneer, the 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.

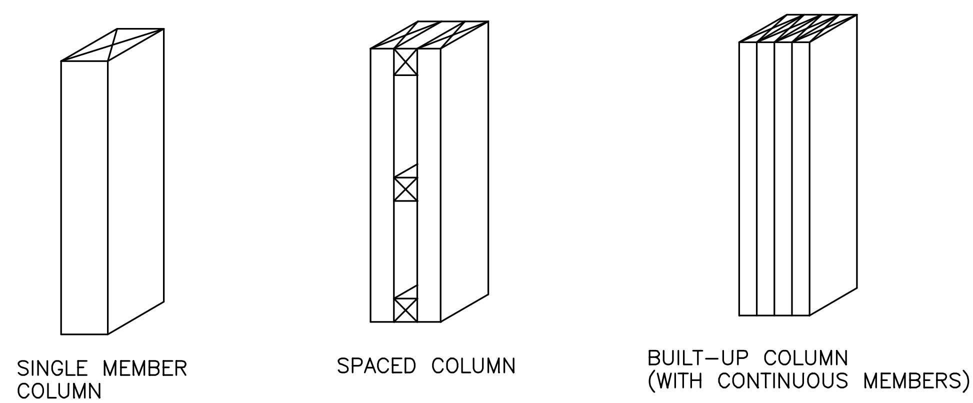
Simple columns are fabricated from one piece of sawn lumber. In residential construction, simple columns such as a 4x4 are common. The equations in Section 5.3 are used to design simple columns as demonstrated in Example 5.8.

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

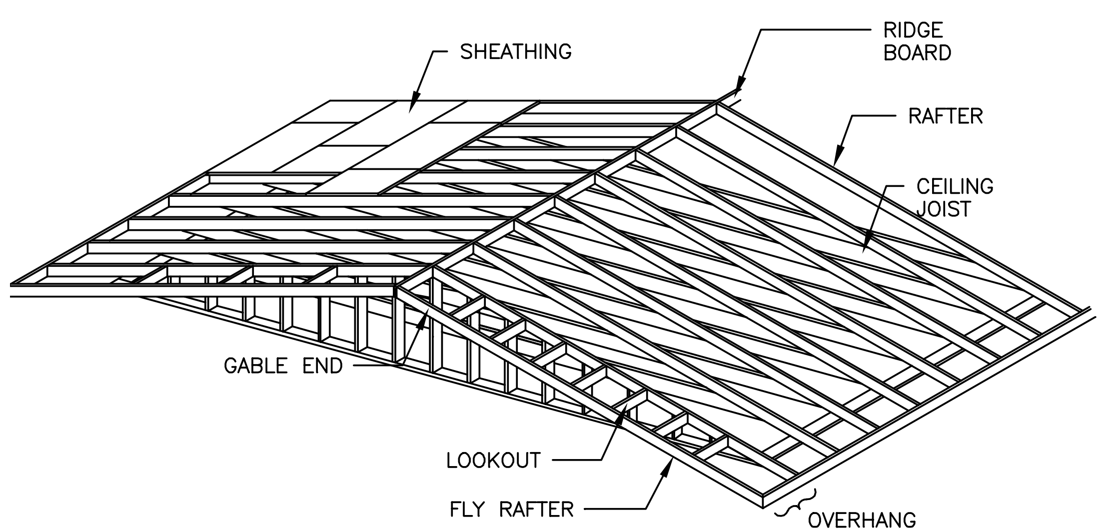
A roof in residential construction is typically a sloped structural system that supports gravity and lateral loads and transfers the loads to the walls below. Generally, the four options for wood roof construction are

- 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.

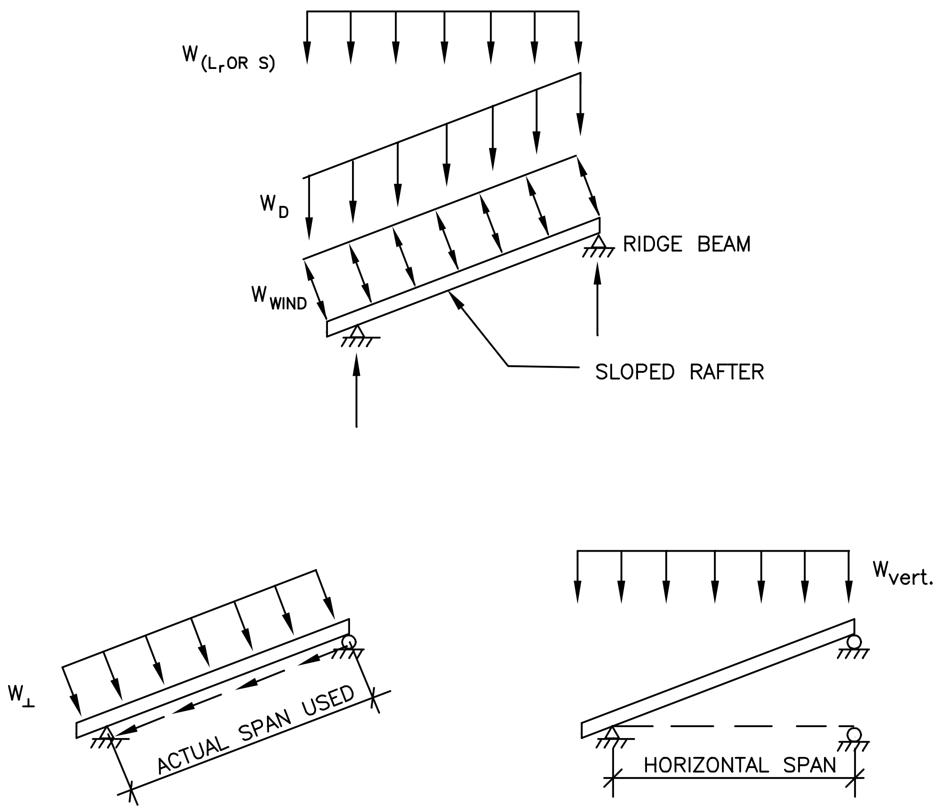
Structural ridge beams are designed to support roof rafters at the ridge when there are no ceiling joists or cross-ties to resist the outward thrust of rafters that would otherwise occur. A repetitive member factor, C_r, is applicable if the ridge beam is composed of two or more members (see Table 5.4). 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*

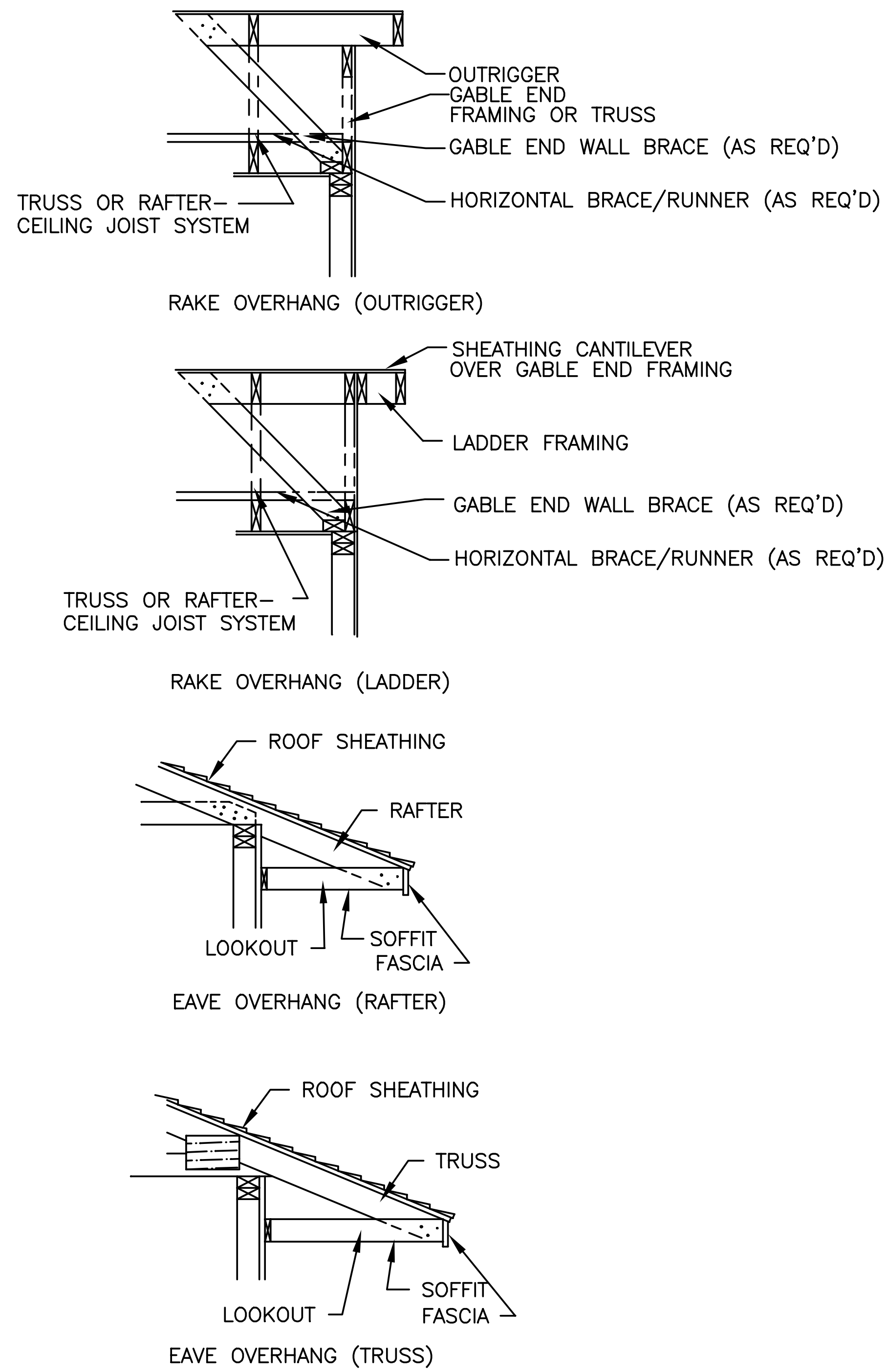


Table 5.6 may assist in dealing with the need to meet a reasonable serviceability limit for deflection (see Section 5.3.2).

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 I_{xx} = 47.63 in^4
F_v = 75 psi S_{xx} = 13.14 in^3
$F_{c\perp}$ = 405 psi b = 1.5 in
E = 1,500,000 psi d = 7.25 in

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

C_D = 1.0 (Section 5.2.4.1)
C_r = 1.15 (Table 5.4)
C_F = 1.2 (NDS-S Table 4A adjustment factors)
C_H = 2.0 (Section 5.2.4.3)
C_L = 1.0 (NDS•3.3.3, continuous lateral support)
C_b = 1.0 (NDS•2.3.10)
F_b' = $F_bC_rC_FC_DC_L$ = 975 (1.15)(1.2)(1.0)(1.0) = 1,345 psi
F_v' = $F_vC_HC_D$ = 75 (2)(1.0) = 150 psi
$F_{c\perp}'$ = $F_{c\perp}C_b$ = 405 (1.0) = 405 psi
E' = E = 1,500,000 psi

3. Calculate the applied load

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

4. Determine maximum clear span based on bending capacity

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

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

$$f_b \leq F_b'$$
$$6.08\,\ell^2 \leq 1{,}345\text{ psi}$$
$$\ell^2 = 221$$
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

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

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(53.3\ \text{plf})^{*}(\ell^4)\ (1{,}728\ \text{in}^3/\text{ft}^3)}{384(1.5\times10^6\ \text{psi})(47.63\ \text{in}^4)} = 1.7 \times 10^{-5}\,\ell^4$$

*applied live load of 40 psf only

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

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

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

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

Conclusion

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

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

d_1 = 7.25 in M_1 = 19% maximum (S-dry lumber)
d_2 = ? M_2 = 10% (estimated equilibrium MC)

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

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

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

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

Given

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

Find Optimum lumber species and grade

Solution

1. Calculate the applied load

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

2. Determine bending stress

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

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

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 $\qquad$ $F_{c\perp}$ = 236 psi
F_v = 39 psi $\qquad$ 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

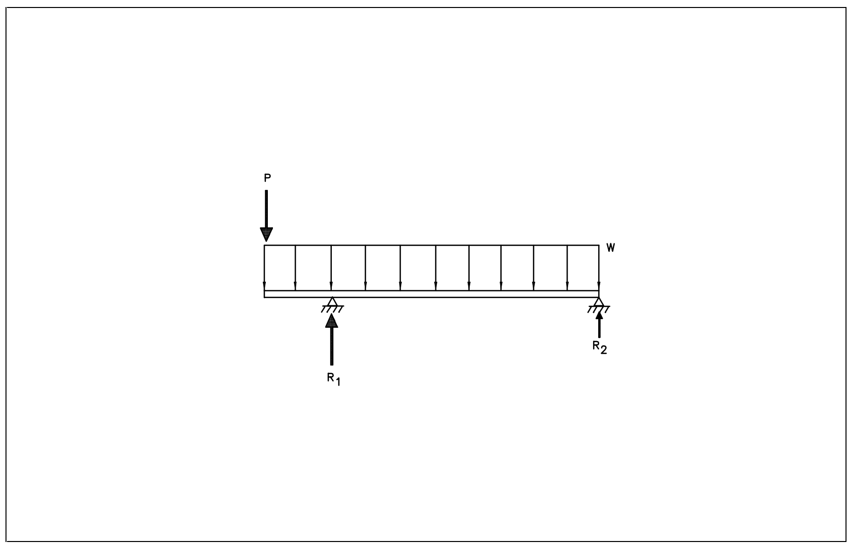
Conclusion

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
F_v = 95 psi
$F_{c\perp}$ = 625 psi
E = 1.7×10^6 psi

S = 21.39 in^3
I = 98.93 in^4
b = 1.5 in
d = 9.25 in

2. Determine lumber property adjustments (see Section 5.2.4)

C_r = 1.15
C_H = 2.0
C_b^* = 1.11

C_F = 1.1
C_D = 1.25 (includes snow)
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).

$F_b' = F_b C_r C_F C_D C_L$ = (1000 psi)(1.15)(1.1)(1.25)(1.0) = 1,581 psi
$F_v' = F_v C_H C_D$ = (95)(2)(1.25) = 238 psi
$F_{c\perp}' = F_{c\perp} C_b$ = 625 (1.11) = 694 psi
E' = E = 1.7×10^6 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)

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

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

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

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

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

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

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

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

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

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

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

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

8. Check bearing stress

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

$$= 207 \text{ psi}$$

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

Conclusion

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

EXAMPLE 5.4 *Built-Up Floor Girder Design*

Given

Loads

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

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

Solution

1. Calculate the design load

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

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

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

3. Lumber property adjustments (Section 5.2.4):

C_r = 1.2 (Table 5.4) $\quad$ C_D = 1.0
C_F = 1.0 $\quad$ C_b = 1.0
C_H = 2.0 $\quad$ 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 psi (1.0)(1.2)(1)(1) = 1,500 psi
$F_V' = F_lC_DC_H$ = 90 psi (1.25)(2.0) = 225 psi
$F_{c\perp}' = F_{c\perp}C_b$ = 565 psi (1) = 565 psi
E' = E = $1.7\text{x}10^6$ psi

4. Determine number of members required due to bending

$$M_{max} = \frac{w\ell^2}{8} = \frac{(600\text{plf})(14\text{ ft})^2}{8} = 14{,}700 \text{ ft-lb}$$

$$f_b = \frac{M}{S} = \frac{(14{,}700 \text{ ft}-\text{lb})(12 \text{ in/ft})}{S} = \frac{176{,}400}{S}$$

$$f_b \leq F_b'$$

$$\frac{176{,}400}{S} \leq 1{,}500 \text{ psi}$$

$$S_x = 118 \text{ in}^3$$

Using Table 1B in NDS-S

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

5. Determine number of members required due to horizontal shear

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

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

$$f_v \le F_v'$$

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

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

2 2x12s A = 33.8 > 28 OK

2 2x10s A = 27.8 ≈ 28 OK

6. Determine required bearing length using 4 2x12s

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

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

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

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

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

7. Determine member size due to deflection

$$\rho_{max} = \frac{5w\ell^4}{384EI} = \frac{5(480\ \text{plf})*(14\ \text{ft})^4\ (1{,}728\ \text{in}^3/\text{ft}^3)}{384\,EI} = \frac{4.15 \times 10^8}{EI}$$

*includes 40 psf live load only

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

$$\rho_{max} \le \rho_{all}$$

$$\frac{4.15 \times 10^8}{EI} = 0.47\ \text{in}$$

$$EI = 8.8 \times 10^8$$

$$(1.7 \times 10^6)(I) = 8.8 \times 10^8$$

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

3 2x12s I = 534 > 519 okay

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

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

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

Try $\ell/480$

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

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

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

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

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

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

Using Table 1B in NDS, use

4 2x12s I = 4 (178 in^4) = 712 in^4 > 706 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

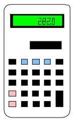
Determine sheathing grade and span rating and thickness by using the APA's *Design and Construction Guide for Residential and Commercial* (APA, 1998). From Table 7 in the APA guide, use 7/16-inch-thick (24/16 rating) sheathing or 15/32-inch- to 1/2-inch-thick (32/16 rating) sheathing. The first number 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.

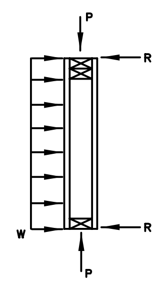
EXAMPLE 5.6 *Exterior Bearing Wall Design*



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 $\qquad$ $F_{c\perp}$ = 425 psi
F_t = 350 psi $\qquad$ F_c = 725 psi
F_v = 70 psi $\qquad$ 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

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

5. Calculate adjusted compressive capacity (NDS•3.7)

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

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

$K_{cE} = 0.3$ visually graded lumber

$c = 0.8$ sawn lumber

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

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

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

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

Axial load only case

Calculations are same as above except use $C_D = 1.25$
$F_c^* = 952$ psi
$C_p = 0.44$
$F_c' = F_cC_DC_FC_P = 725\text{ psi }(1.25)(1.05)(0.44) = 419$ 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 = $(856 \text{ lb/stud})\left(\dfrac{1 \text{ stud}}{2 \text{ ft}}\right)$ = 428 plf (uniform dead load at top of wall)

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

7. Determine maximum axial gravity load without bending load

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

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

$F_c \leq F_c'$
$\dfrac{P}{A} \leq 419 \text{ psi}$
P_{max} = (419 psi)(1.5 in)(3.5 in) = 2,200 lbs/stud

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

8. Check bearing capacity of wall plate

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

Conclusion

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

Note:

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

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

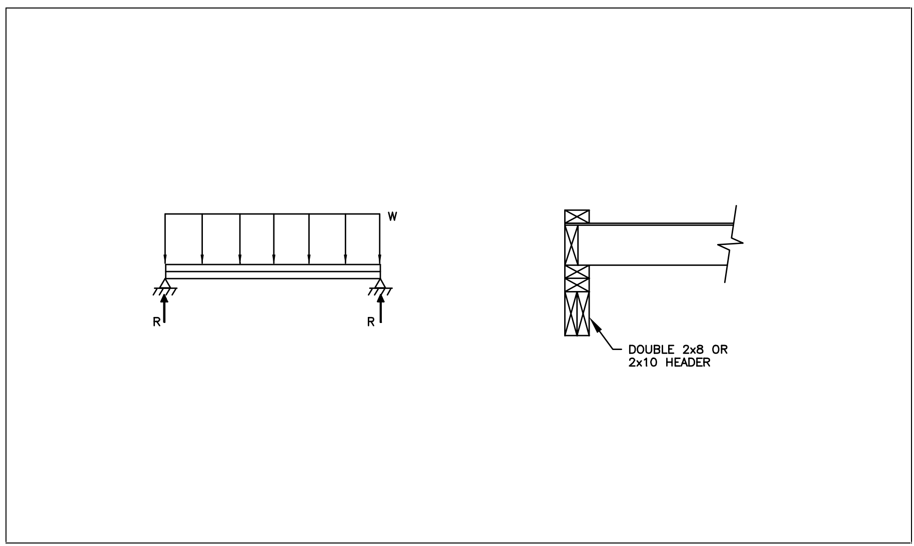
EXAMPLE 5.7 *Header System Design*

Given

Two-story house

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

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



Header System

Solution

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

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

2. Determine lumber property adjustments (Section 5.2.4)

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

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

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

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

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

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

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

$F_c' = F_cC_DC_FC_p = (1{,}150 \text{ psi})(1.0)(1.15)(0.44) = 582 \text{ psi}$
$P_{all} = F'_cA = (582 \text{ psi})(3.5 \text{ in})(3.5 \text{ in}) = 7{,}129 \text{ lb} > 4{,}800 \text{ lb}$
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

$$\ell_e = 1.63\,\ell_u + 3d$$
$$= 1.63\ (14.4\ ft) + 3\ (7.25\ in)(1\ in/12ft)$$
$$= 25.3\ ft$$
$$R_B = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{(25.5\ ft)(12\ in/ft)(7.25\ in)}{(1.5\ in)^2}}$$
$$= 31 \quad < \quad 50\ (OK)$$
$$K_{bE} = 0.439 \quad \text{(visually graded lumber)}$$
$$F_{bE} = \frac{K_{bE} E'}{R_B^{\,2}} = \frac{0.439\,(1.6x10^6\ psi)}{(31)^2} = 730\ psi$$
$$F_b{*} = F_b C_D C_r C_F$$
$$= 900\ psi\ (1.6)(1.15)(1.2) = 1{,}987\ psi$$
$$C_L = \frac{1+(F_{bE}/F_b{*})}{1.9} - \sqrt{\left[\frac{1+(F_{bE}/F_b{*})}{1.9}\right]^2 - \frac{F_{bE}/F_b{*}}{0.95}}$$
$$C_L = 0.36 \quad (2x8)$$

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

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

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

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

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)

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

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

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

$\rho_{max} << \rho_{all}$ (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 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} = $1/8\ w\ell^2$ = $1/8\ (49\ \text{plf})(13\text{ft})^2$
= 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} = $1/8\ (469\ \text{plf})(13\ \text{ft})^2$ = 9,908 ft-lb

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

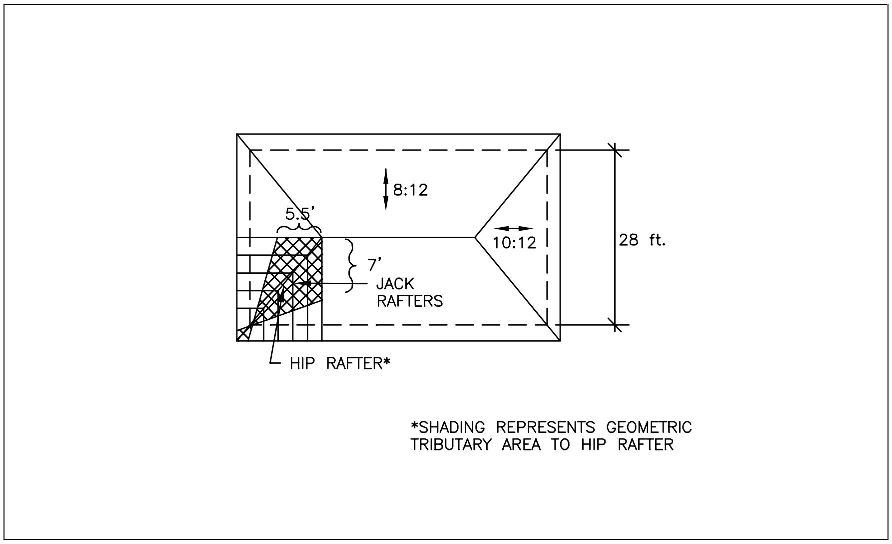
Note. The remainder of the problem is essentially identical to Example 5.9 with respect to determining the strength of the wood member. However, a trial member size and grade are needed to determine the lumber stresses as well as the lumber property adjustment values. Thus, the process of optimizing 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:

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

$$f_v << F_v'$$

OK by inspection

h. Consider deflection:

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

Conclusion

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

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

CHAPTER 6

Lateral Resistance to Wind and Earthquake

6.1 General

The objectives in designing a building's lateral resistance to wind and earthquake forces are

- to provide a system of shear walls, diaphragms, and interconnections to transfer lateral loads and overturning forces to the foundation;
- to prevent building collapse in extreme wind and seismic events; and
- to provide adequate stiffness to the structure for service loads experienced in moderate wind and seismic events.

In light-frame construction, the lateral force-resisting system (LFRS) comprises shear walls, diaphragms, and their interconnections to form a whole-building system that may behave differently than the sum of its individual parts. In fact, shear walls and diaphragms are themselves subassemblies of many parts and connections. Thus, designing an efficient LFRS system is perhaps the greatest challenge in the structural design of light-frame buildings. In part, the challenge results from the lack of any single design methodology or theory that provides reasonable predictions of complex, large-scale system behavior in conventionally built or engineered light-frame buildings.

Designer judgment is a crucial factor that comes into play when the designer selects how the building is to be analyzed and to what extent the analysis should be assumed to be a correct representation of the true design problem. Designer judgment is essential in the early stages of design because the analytic methods and assumptions used to evaluate the lateral resistance of light-frame buildings are not in themselves correct representations of the problem. They are analogies that are sometimes reasonable but at other times depart significantly from reason and actual system testing or field experience.

This chapter focuses on methods for evaluating the lateral resistance of individual subassemblies of the LFRS (i.e., shear walls and diaphragms) and the response of the whole building to lateral loads (i.e., load distribution). Traditional design approaches as well as innovative methods, such as the *perforated shear wall design method*, are integrated into the designer's “tool box.” While the code-approved methods have generally “worked,” there is considerable opportunity for improvement and optimization. Therefore, the information and design examples presented in this chapter provide a useful guide and resource that supplement existing building code provisions. More important, the chapter is aimed at fostering a better understanding of the role of analysis versus judgment and promoting more efficient design in the form of alternative methods.

The lateral design of light-frame buildings is not a simple endeavor that provides “exact” solutions. By the very nature of the LFRS, the real behavior of light-frame buildings is highly dependent on the performance of building systems, including the interactions of structural and nonstructural components. For example, the nonstructural components in conventional housing (i.e., sidings, interior finishes, interior partition walls, and even windows and trim) can account for more than 50 percent of a building’s lateral resistance. Yet, the contribution of these components is not considered as part of the “designed” LFRS for lack of appropriate design tools and building code provisions that may prohibit such considerations. In addition, the need for simplified design methods inevitably leads to a trade-off–analytical simplicity for design efficiency.

In seismic design, factors that translate into better performance may not always be obvious. The designer should become accustomed to thinking in terms of the relative stiffness of components that make up the whole building. Important, too, is an understanding of the inelastic (nonlinear), nonrigid body behavior of wood-framed systems that affect the optimization of strength, stiffness, dampening, and ductility. In this context, the concept that more strength is better is insupportable without considering the impact on other important factors. Many factors relate to a structural system’s deformation capability and ability to absorb and safely dissipate energy from abusive cyclic motion in a seismic event. The intricate interrelationship of these several factors is difficult to predict with available seismic design approaches.

For example, the basis for the seismic response modifier R is a subjective representation of the behavior of a given structure or structural system in a seismic event (refer to Chapter 3). In a sense, it bears evidence of the inclusion of “fudge factors” in engineering science for reason of necessity (not of preference) in attempting to mimic reality. It is not necessarily surprising, then, that the amount of wall bracing in conventional homes shows no apparent correlation with the damage levels experienced in seismic events (HUD, 1999). Similarly, the near-field damage to conventional homes in the Northridge Earthquake did not correlate with the magnitude of response spectral ground accelerations in the short period range (HUD, 1999). The short-period spectral response acceleration, it will be recalled, is the primary ground motion parameter used in the design of most low-rise and light-frame buildings (refer to Chapter 3).

The apparent lack of correlation between design theory and actual outcome points to the tremendous uncertainty in existing seismic design methods for light-frame structures. In essence, a designer's compliance with accepted seismic design provisions may not necessarily be a good indication of actual performance in a major seismic event. This statement may be somewhat unsettling but is worthy of mention. For wind design, the problem is not as severe in that the lateral load can be more easily treated as a static load, with system response primarily a matter of determining lateral capacity without complicating inertial effects, at least for small light-frame buildings.

In conclusion, the designer should have a reasonable knowledge of the underpinnings of current LFRS design approaches (including their uncertainties and limitations). However, many designers do not have the opportunity to become familiar with the experience gained from testing whole buildings or assemblies. Design provisions are generally based on an "element-based" approach to engineering and usually provide little guidance on the performance of the various elements as assembled in a real building. Therefore, the next section presents a brief overview of several whole-house lateral load tests.

6.2 Overview of Whole-Building Tests

A growing number of full-scale tests of houses have been conducted to gain insight into actual system strength and structural behavior. Several researchers have recently summarized the body of research; the highlights follow (Thurston, 1994; NIST, 1998).

One whole-house test program investigated the lateral stiffness and natural frequency of a production-built home (Yokel, Hsi, and Somes, 1973). The study applied a design load simulating a uniform wind pressure of 25 psf to a conventionally built home: a two-story, split-foyer dwelling with a fairly typical floor plan. The maximum deflection of the building was only 0.04 inches and the residual deflection about 0.003 inches. The natural frequency and dampening of the building were 9 hz (0.11 s natural period) and 6 percent, respectively. The testing was nondestructive such that the investigation yielded no information on "postyielding" behavior; however, the performance was good for the nominal lateral design loads under consideration.

Another whole-house test applied transverse loads without uplift to a wood-framed house. Failure did not occur until the lateral load reached the "equivalent" of a 220 mph wind event without inclusion of uplift loads (Tuomi and McCutcheon, 1974). The house was fully sheathed with 3/8-inch plywood panels, and the number of openings was somewhat fewer than would be expected for a typical home (at least on the street-facing side). The failure took the form of slippage at the floor connection to the foundation sill plate (i.e., there was only one 16d toenail at the end of each joist, and the band joist was not connected to the sill). The connection was somewhat less than what is now required in the United States for conventional residential construction (ICC, 1998). The racking stiffness of the walls nearly doubled from that experienced before the addition of the roof framing. In addition, the simple 2x4 wood trusses were able to carry a gravity load of 135 psf–more than three times the design load of 40 psf. However, it is important to note that combined uplift and lateral load, as would be expected in high-wind conditions, was not tested. Further, the test house was relatively small and "boxy" in comparison to modern homes.

Many whole-house tests have been conducted in Australia. In one series of whole-house tests, destructive testing has shown that conventional residential construction (only slightly different from that in the United States) was able to withstand 2.4 times its intended design wind load (corresponding to a 115 mph wind speed) without failure of the structure (Reardon and Henderson, 1996). The test house had typical openings for a garage, doors, and windows, and no special wind-resistant detailing. The tests applied a simultaneous roof uplift load of 1.2 times the total lateral load. The drift in the two-story section was 3 mm at the maximum applied load while the drift in the open one-story section (i.e., no interior walls) was 3 mm at the design load and 20 mm at the maximum applied load.

Again in Australia, a house with fiber cement exterior cladding and plasterboard interior finishes was tested to 4.75 times its “design” lateral load capacity (Boughton and Reardon, 1984). The walls were restrained with tie rods to resist wind uplift loads as required in Australia’s typhoon-prone regions. The roof and ceiling diaphragm was found to be stiff; in fact, the diaphragm rigidly distributed the lateral loads to the walls. The tests suggested that the house had sufficient capacity to resist a design wind speed of 65 m/s (145 mph).

Yet another Australian test of a whole house found that the addition of interior ceiling finishes reduced the deflection (i.e., drift) of one wall line by 75 percent (Reardon, 1988; Reardon, 1989). When cornice trim was added to cover or dress the wall-ceiling joint, the deflection of the same wall was reduced by another 60 percent (roughly 16 percent of the original deflection). The tests were conducted at relatively low load levels to determine the impact of various nonstructural components on load distribution and stiffness.

Recently, several whole-building and assembly tests in the United States have been conducted to develop and validate sophisticated finite-element computer models (Kasal, Leichti, and Itani, 1994). Despite some advances in developing computer models as research tools, the formulation of a simplified methodology for application by designers lags behind. Moreover, the computer models tend to be time-intensive to operate and require detailed input for material and connection parameters that would not normally be available to typical designers. Given the complexity of system behavior, the models are often not generally applicable and require “recalibration” whenever new systems or materials are specified.

In England, researchers have taken a somewhat different approach by moving directly from empirical system data to a simplified design methodology, at least for shear walls (Griffiths and Wickens, 1996). This approach applies various “system factors” to basic shear wall design values to obtain a value for a specific application. System factors account for material effects in various wall assemblies, wall configuration effects (i.e., number of openings in the wall), and interaction effects with the whole building. One factor even accounts for the fact that shear loads on wood-framed shear walls in a full brick-veneered building are reduced by as much as 45 percent for wind loads, assuming, of course, that the brick veneer is properly installed and detailed to resist wind pressures.

More recently, whole-building tests have been conducted in Japan (and to a lesser degree in the United States) by using large-scale shake tables to study the inertial response of whole, light-frame buildings (Yasumura, 1999). The tests have demonstrated whole-building stiffness of about twice that experienced by walls tested independently. The results are reasonably consistent with those reported above. Apparently, many whole-building tests have been conducted in Japan, but the associated reports are available only in Japanese (Thurston, 1994).

The growing body of whole-building test data will likely improve the understanding of the actual performance of light-frame structures in seismic events to the extent that the test programs are able to replicate actual conditions. Actual performance must also be inferred from anecdotal experience or, preferably, from experimentally designed studies of buildings experiencing major seismic or wind events (refer to Chapter 1).

6.3 LFRS Design Steps and Terminology

The lateral force resisting system (LFRS) of a home is the “whole house” including practically all structural and non-structural components. To enable a rational and tenable design analysis, however, the complex structural system of a light-frame house is usually subjected to many simplifying assumptions; refer to Chapter 2. The steps required for thoroughly designing a building’s LFRS are outlined below in typical order of consideration:

1. Determine a building’s architectural design, including layout of walls and floors (usually pre-determined).
2. Calculate the lateral loads on the structure resulting from wind and/or seismic conditions (refer to Chapter 3).
3. Distribute shear loads to the LFRS (wall, floor, and roof systems) based on one of the design approaches described later in this chapter (refer to Section 6.4.1).
4. Determine *shear wall* and *diaphragm* assembly requirements for the various LFRS components (sheathing thickness, fastening schedule, etc.) to resist the stresses resulting from the applied lateral forces (refer to Section 6.5).
5. Design the *hold-down restraints* required to resist overturning forces generated by lateral loads applied to the vertical components of the LFRS (i.e., shear walls).
6. Determine interconnection requirements to transfer shear between the LFRS components (i.e., roof, walls, floors, and foundation).
7. Evaluate *chords* and *collectors* (or *drag struts*) for adequate capacity and for situations requiring special detailing such as splices.

It should be noted that, depending on the method of distributing shear loads (refer to Section 6.4.1), Step 3 may be considered a preliminary design step. If, in fact, loads are distributed according to stiffness in Step 3, then the LFRS must already be defined; therefore, the above sequence can become iterative between Steps 3 and 4. A designer need not feel compelled to go to such a level of complexity (i.e., using a stiffness-based force distribution) in designing a simple home, but the decision becomes less intuitive with increasing plan complexity.

The above list of design steps introduced several terms that are defined below.

Horizontal diaphragms are assemblies such as the roof and floors that act as "deep beams" by collecting and transferring lateral forces to the *shear walls*, which are the vertical components of the LFRS. The diaphragm is analogous to a horizontal, simply supported beam laid flatwise; a shear wall is analogous to a vertical, fixed-end, cantilevered beam. Chapter 2 discussed the function of the LFRS and the lateral load path. The reader is referred to that chapter for a conceptual overview of the LFRS and to Chapter 3 for methodologies to calculate lateral loads resulting from wind and earthquake forces.

Chords are the members (or a system of members) that form a "flange" to resist the tension and compression forces generated by the "beam" action of a diaphragm or shear wall. As shown in Figure 6.1, the chord members in shear walls and diaphragms are different members, but they serve the same purpose in the beam analogy. A *collector* or *drag strut*, which is usually a system of members in light-frame buildings, "collects" and transfers loads by tension or compression to the shear resisting segments of a wall line (see Figure 6.2a).

In typical light-frame homes, special design of chord members for floor diaphragms may involve some modest detailing of splices at the diaphragm boundary (i.e., joints in the band joists). If adequate connection is made between the band joist and the wall top plate, then the diaphragm sheathing, band joists, and wall framing function as a "composite" chord in resisting the chord forces. Thus, the diaphragm chord is usually integral with the collectors or drag struts in shear walls. Given that the collectors on shear walls often perform a dual role as a chord on a floor or roof diaphragm boundary, the designer needs only to verify that the two systems are reasonably interconnected along their boundary, thus ensuring composite action as well as direct shear transfer (i.e., slip resistance) from the diaphragm to the wall. As shown in Figure 6.2b, the failure plane of a typical "composite" collector or diaphragm chord can involve many members and their interconnections.

For shear walls in typical light-frame buildings, tension and compression forces on shear wall chords are usually considered. In particular, the connection of hold-downs to shear wall chords should be carefully evaluated with respect to the transfer of tension forces to the structure below. Tension forces result from the overturning action (i.e., overturning moment) caused by the lateral shear load on the shear wall. In some cases, the chord may be required to be a thicker member to allow for an adequate hold-down connection or to withstand the tension and compression forces presumed by the beam analogy. Fortunately, most chords in light-frame shear walls are located at the ends of walls or adjacent to openings where multiple studs are already required for reasons of constructability and gravity load resistance (see cross-section "B" in Figure 6.1).

FIGURE 6.1 *Chords in Shear Walls and Horizontal Diaphragms Using the "Deep Beam" Analogy*

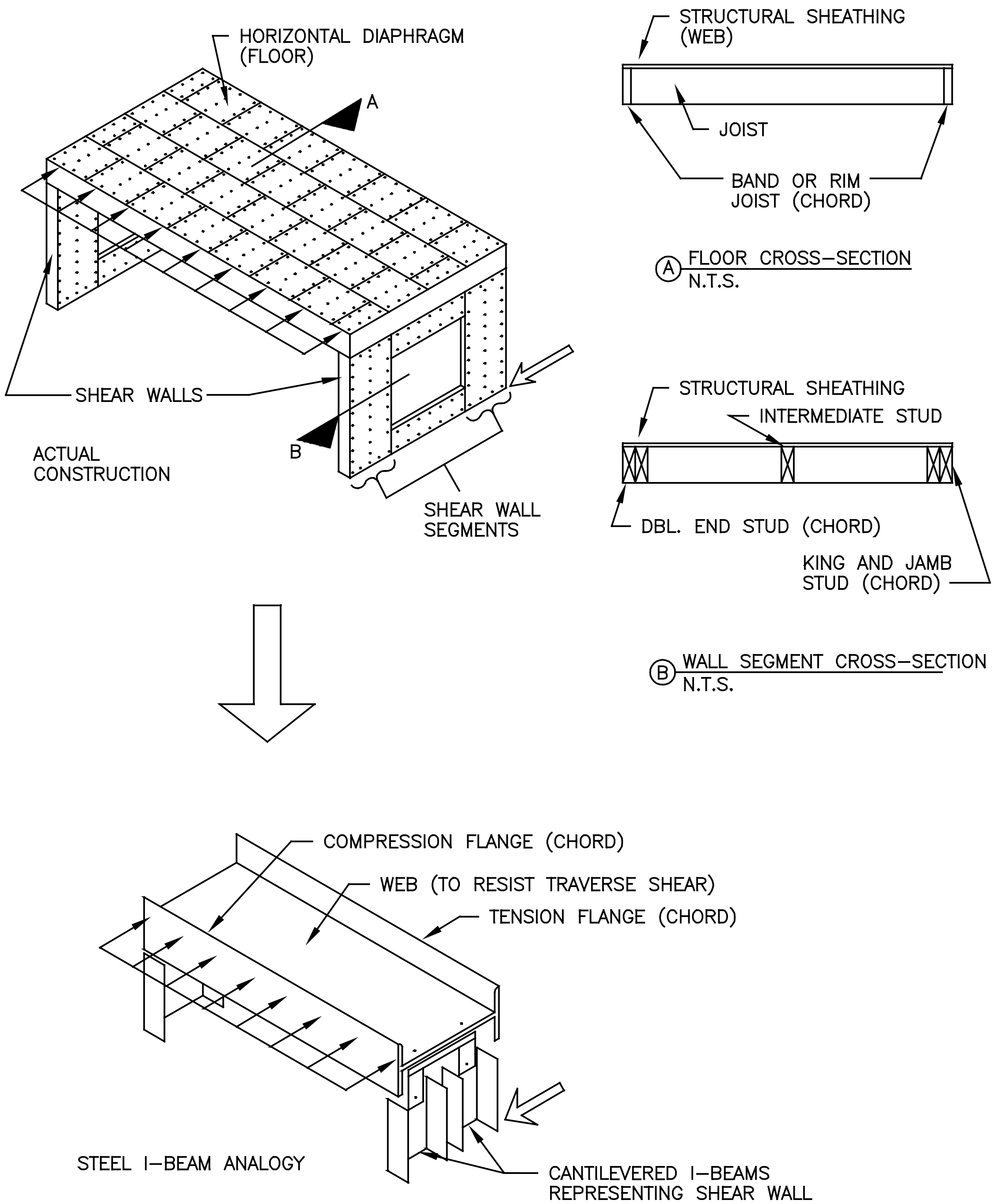
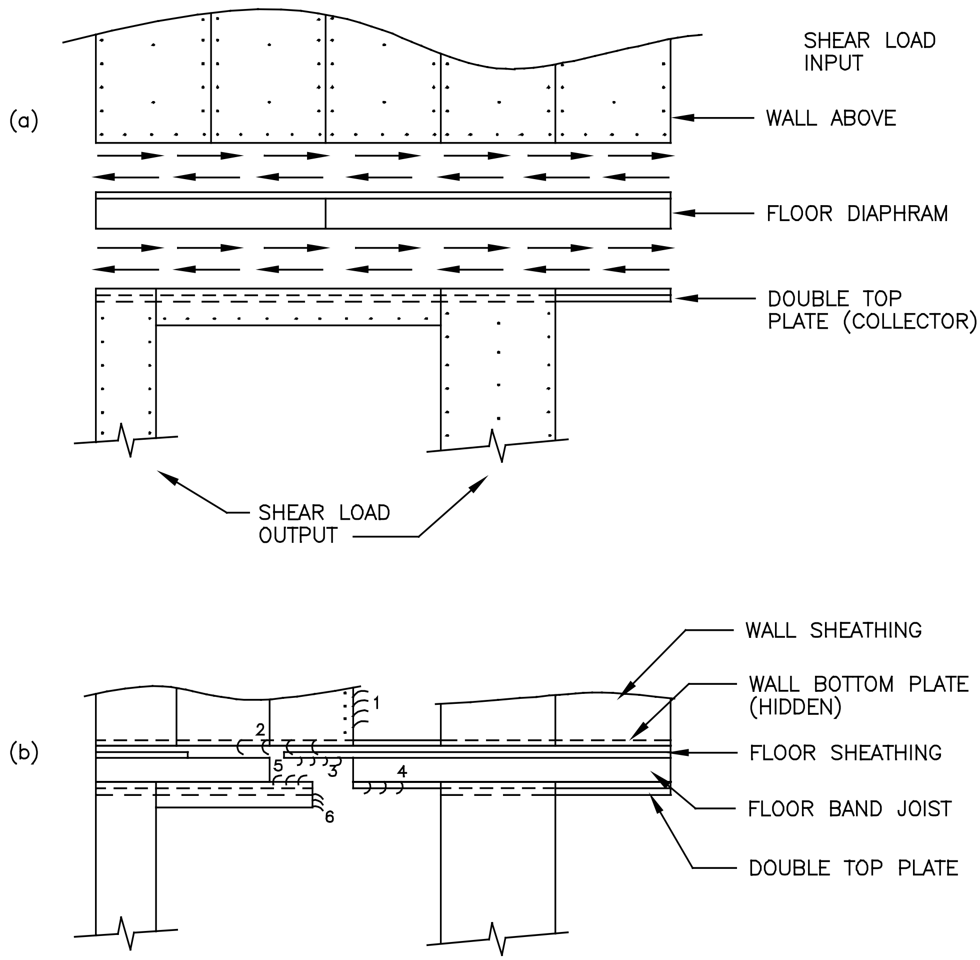


FIGURE 6.2 ***Shear Wall Collector and the Composite Failure Plane (Failure plane also applies to diaphragm chords)***



COMPOSITE FAILURE PLANE FOR A COLLECTOR:

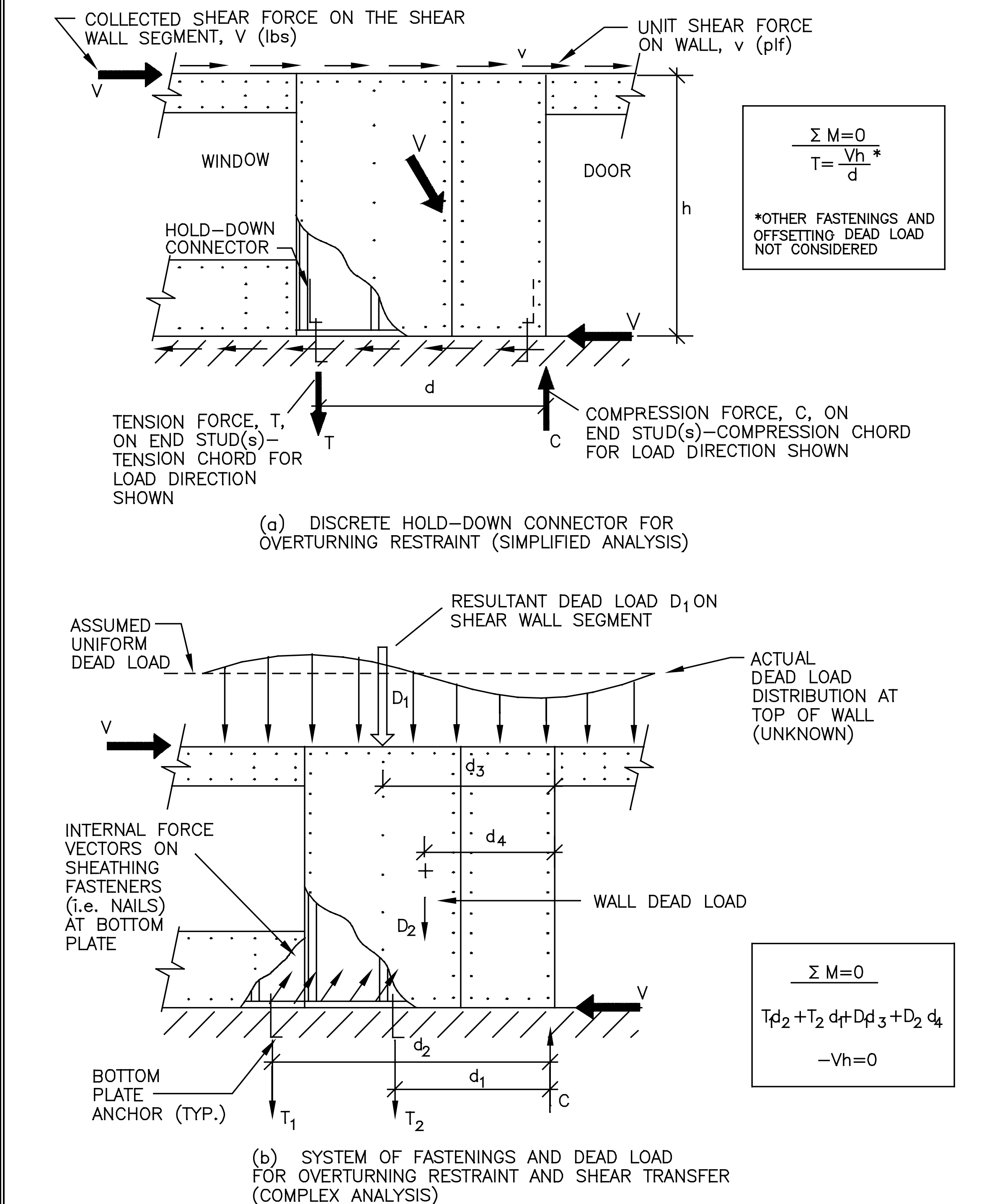
1. SHEATHING FASTENERS AT COMMON WALL STUD
2. WALL BOTTOM PLATE FASTENERS
3. FLOOR SHEATHING FASTENERS
4. DOUBLE TOP PLATE SPLICE NAILS
5. SHEATHING FASTENERS AT REGION OF DOUBLE TOP PLATE SPLICE
6. SHEATHING AND HEADER FASTENERS TO KING STUD

Hold-down restraints are devices used to restrain the whole building and individual shear wall segments from the overturning that results from the leveraging (i.e., overturning moment) created by lateral forces. The current engineering approach calls for restraints that are typically metal connectors (i.e., straps or brackets) that attach to and anchor the chords (i.e., end studs) of shear wall segments (see Figure 6.3a). In many typical residential applications, however, overturning forces may be resisted by the dead load and the contribution of many component connections (see Figure 6.3b). Unfortunately (but in reality), this consideration may require a more intensive analytic effort and greater degree of designer presumption because overturning forces may disperse through many “load paths” in a nonlinear fashion. Consequently, the analysis of overturning becomes much more complicated; the designer cannot simply assume a single load path through a single hold-down connector. Indeed, analytic knowledge of overturning has not matured sufficiently to offer an exact performance-based solution, even though experience suggests that the resistance provided by conventional framing has proven adequate to prevent collapse in all but the most extreme conditions or mis-applications (see Chapter 1 and Section 6.2).

Framing and fastenings at wall corner regions are a major factor in explaining the actual behavior of conventionally built homes, yet there is no currently recognized way to account for this effect from a performance-based design perspective. Several studies have investigated corner framing effects in restraining shear walls without the use of hold-down brackets. In one such study, cyclic and monotonic tests of typical 12-foot-long wood-framed shear walls with 2- and 4-foot corner returns have demonstrated that overturning forces can be resisted by reasonably detailed corners (i.e., sheathing fastened to a common corner stud), with the reduction in shear capacity only about 10 percent from that realized in tests of walls with hold-downs instead of corner returns (Dolan and Heine, 1997c). The corner framing approach can also improve ductility (Dolan and Heine, 1997c) and is confirmed by testing in other countries (Thurston, 1994). In fact, shear wall test methods in New Zealand use a simple three-nail connection to provide hold-down restraint (roughly equivalent to three 16d common nails in a single shear wood-to-wood connection with approximately a 1,200- to 1,500-pound ultimate capacity). The three-nail connection resulted from an evaluation of the restraining effect of corners and the selection of a minimum value from typical construction. The findings of the tests reported above do not consider the beneficial contribution of the dead load in helping to restrain a corner from uplift as a result of overturning action.

The discussion to this point has given some focus to conventional residential construction practices for wall bracing that have worked effectively in typical design conditions. This observation is a point of contention, however, because conventional construction lacks the succinct loads paths that may be assumed when following an accepted engineering method. Therefore, conventional residential construction does not lend itself readily to current engineering conventions of analyzing a lateral force resisting system in light-frame construction. As a result, it is difficult to define appropriate limitations to the use of conventional construction practices based purely on existing conventions of engineering analysis.

FIGURE 6.3 ***Two Types of Hold-Down Restraint and Basic Analytic Concepts***



6.4 The Current LFRS Design Practice

This section provides a brief overview of the current design practices for analyzing the LFRS of light-frame buildings. It highlights the advantages and disadvantages of the various approaches but, in the absence of a coherent body of evidence, makes no attempt to identify which approach, if any, may be considered superior. Where experience from whole-building tests and actual building performance in real events permits, the discussion provides a critique of current design practices that, for lack of better methods, relies somewhat on an intuitive sense for the difference between the structure as it is analyzed and the structure as it may actually perform. The intent is not to downplay the importance of engineering analysis; rather, the designer should understand the implications of the current analytic methods and their inherent assumptions and then put them into practice in a suitable manner.

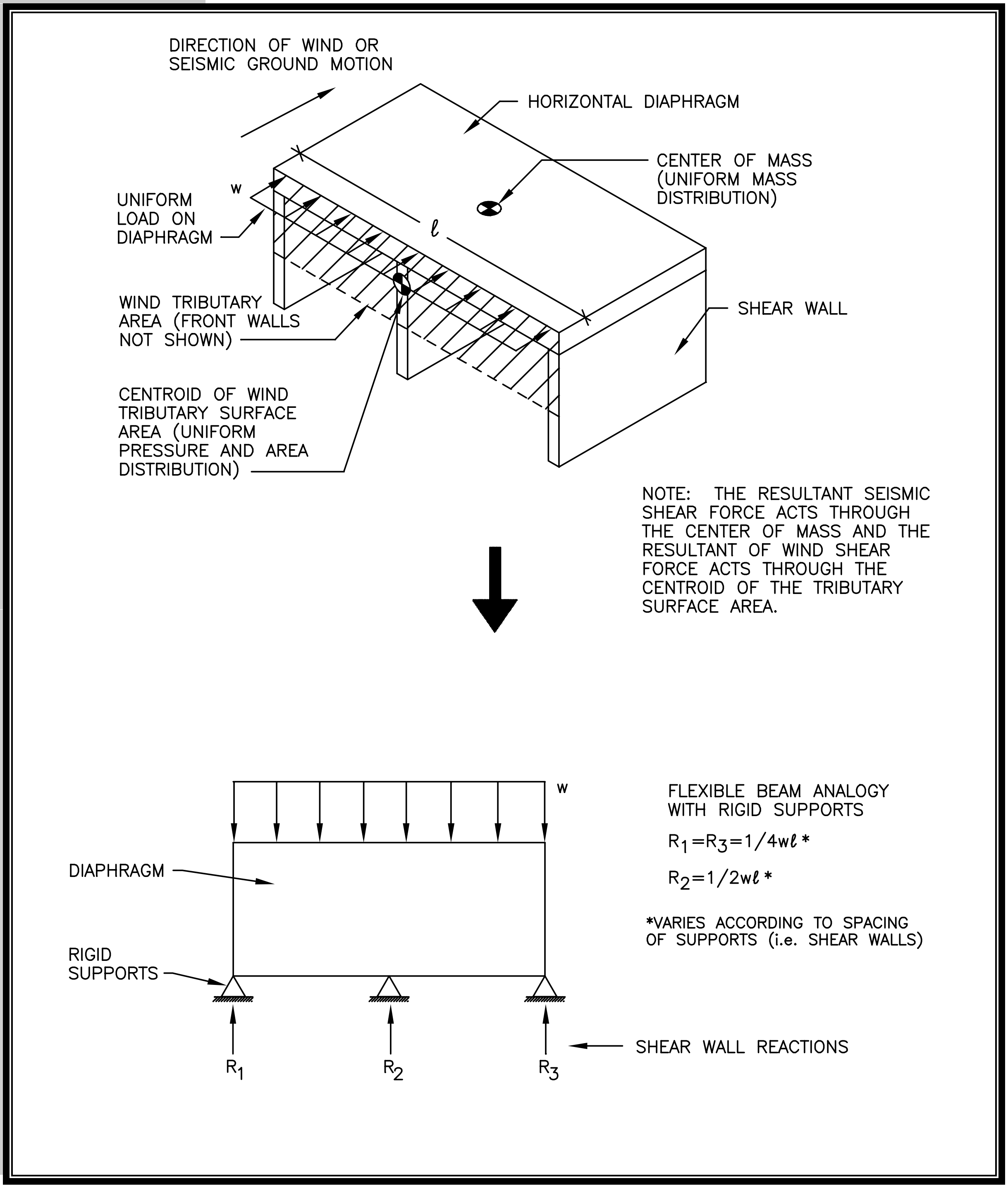
6.4.1 Lateral Force Distribution Methods

The design of the LFRS of light-frame buildings generally follows one of three methods described below. Each differs in its approach to distributing whole-building lateral forces through the horizontal diaphragms to the shear walls. Each varies in the level of calculation, precision, and dependence on designer judgment. While different solutions can be obtained for the same design by using the different methods, one approach is not necessarily preferred to another. All may be used for the distribution of seismic and wind loads to the shear walls in a building. However, some of the most recent building codes may place limitations or preferences on certain methods.

Tributary Area Approach (Flexible Diaphragm)

The *tributary area approach* is perhaps the most popular method used to distribute lateral building loads. Tributary areas based on building geometry are assigned to various components of the LFRS to determine the wind or seismic loads on building components (i.e., shear walls and diaphragms). The method assumes that a diaphragm is relatively flexible in comparison to the shear walls (i.e., a “flexible diaphragm”) such that it distributes forces according to tributary areas rather than according to the stiffness of the supporting shear walls. This hypothetical condition is analogous to conventional beam theory, which assumes rigid supports as illustrated in Figure 6.4 for a continuous horizontal diaphragm (i.e., floor) with three supports (i.e., shear walls).

FIGURE 6.4 ***Lateral Force Distribution by a "Flexible" Diaphragm (tributary area approach)***



In seismic design, tributary areas are associated with uniform area weights (i.e., dead loads) assigned to the building systems (i.e., roof, walls, and floors) that generate the inertial seismic load when the building is subject to lateral ground motion (refer to Chapter 3 on earthquake loads). In wind design, the tributary areas are associated with the lateral component of the wind load acting on the exterior surfaces of the building (refer to Chapter 3 on wind loads).

The flexibility of a diaphragm depends on its construction as well as on its aspect ratio (length:width). Long, narrow diaphragms, for example, are more flexible in bending along the their long dimension than short, wide diaphragms. In other words, rectangular diaphragms are relatively stiff in one loading direction and relatively flexible in the other. Similarly, long shear walls with few openings are stiffer than walls comprised of only narrow shear wall segments. While analytic methods are available to calculate the stiffness of shear wall segments and diaphragms (refer to Section 6.5), the actual stiffness of these systems is extremely difficult to predict accurately (refer to Section 6.2). It should be noted that if the diaphragm is considered infinitely rigid relative to the shear walls and the shear walls have roughly equivalent stiffness, the three shear wall reactions will be roughly equivalent (i.e., $R_1 = R_2 = R_3 = 1/3[w][l]$). If this assumption were more accurate, the interior shear wall would be overdesigned and the exterior shear walls underdesigned with use of the tributary area method. In many cases, the correct answer is probably somewhere between the apparent over- and under-design conditions.

The tributary area approach is reasonable when the layout of the shear walls is generally symmetrical with respect to even spacing and similar strength and stiffness characteristics. It is particularly appropriate in concept for simple buildings with diaphragms supported by two exterior shear wall lines (with similar strength and stiffness characteristics) along both major building axes. More generally, the major advantages of the tributary area LFRS design method are its simplicity and applicability to simple building configurations. In more complex applications, the designer should consider possible imbalances in shear wall stiffness and strength that may cause or rely on torsional response to maintain stability under lateral load (see *relative stiffness design approach*).

Total Shear Approach ("Eyeball" Method)

Considered the second most popular and simplest of the three LFRS design methods, the *total shear approach* uses the total story shear to determine a total amount of shear wall length required on a given story level for each orthogonal direction of loading. The amount of shear wall is then "evenly" distributed in the story according to designer judgment. While the total shear approach requires the least amount of computational effort among the three methods, it demands good "eyeball" judgment as to the distribution of the shear wall elements in order to address or avoid potential loading or stiffness imbalances. In seismic design, loading imbalances may be created when a building's mass distribution is not uniform. In wind design, loading imbalances result when the surface area of the building is not uniform (i.e., taller walls or steeper roof sections experience greater lateral wind load). In both cases, imbalances are created when the center of resistance is offset from either the center of mass (seismic design) or the resultant force center of the exterior surface pressures (wind design). Thus, the reliability of the total shear approach is highly dependent on the designer's judgment and intuition regarding load distribution and structural response. If used indiscriminately without consideration of the above factors, the total shear approach to LFRS design can result in poor performance in severe seismic or wind events. However, for small structures such as homes, the method has produced reasonable designs, especially in view of the overall uncertainty in seismic and wind load analysis.

Relative Stiffness Design Approach (Rigid Diaphragm)

The *relative stiffness approach* was first contemplated for house design in the 1940s and was accompanied by an extensive testing program to create a database of racking stiffnesses for a multitude of interior and exterior wall constructions used in residential construction at that time (NBS, 1948). If the horizontal diaphragm is considered stiff relative to the shear walls, then the lateral forces on the building are distributed to the shear wall lines according to their relative stiffness. A stiff diaphragm may then rotate some degree to distribute loads to all walls in the building, not just to walls parallel to an assumed loading direction. Thus, the relative stiffness approach considers torsional load distribution as well as distribution of the direct shear loads. When torsional force distribution needs to be considered, whether to demonstrate lateral stability of an "unevenly" braced building or to satisfy a building code requirement, the relative stiffness design approach is the only available option.

Although the approach is conceptually correct and comparatively more rigorous than the other two methods, its limitations with respect to reasonably determining the real stiffness of shear wall lines (composed of several restrained and unrestrained segments and nonstructural components) and diaphragms (also affected by nonstructural components and the building plan configuration) render its analogy to actual structural behavior uncertain. Ultimately, it is only as good as the assumptions regarding the stiffness or shear walls and diaphragms relative to the actual stiffness of a complete building system. As evidenced in the previously mentioned whole-building tests and in other authoritative design texts on the subject (Ambrose and Vergun, 1987), difficulties in accurately predicting the stiffness of shear walls and diaphragms in actual buildings are significant. Moreover, unlike the other methods, the relative stiffness design approach is iterative in that the distribution of loads to the shear walls requires a preliminary design so that relative stiffness may be estimated. One or more adjustments and recalculations may be needed before reaching a satisfactory final design.

However, it is instructional to consider analytically the effects of stiffness in the distribution of lateral forces in an LFRS, even if based on somewhat idealized assumptions regarding relative stiffness (i.e., diaphragm is rigid over the entire expanse of shear walls). The approach is a reasonable tool when the torsional load distribution should be considered in evaluating or demonstrating the stability of a building, particularly a building that is likely to undergo significant torsional response in a seismic event. Indeed, torsional imbalances exist in just about any building and may be responsible for the relatively good performance of some light-frame homes when one side (i.e., the street-facing side of the building) is weaker (i.e., less stiff and less strong) than the other three sides of the building. This condition is common owing to the aesthetic desire and functional need for more openings on the front side of a building. However, a torsional response in the case of underdesign (i.e., "weak" or "soft" story) can wreak havoc on a building and constitute a serious threat to life.

6.4.2 Shear Wall Design Approaches

Once the whole-building lateral loads have been distributed and assigned to the floor and roof diaphragms and various designated shear walls, each of these subassemblies must be designed to resist the assigned shear loads. As discussed, the whole-building shear loads are distributed to various shear walls ultimately in accordance with the principle of relative stiffness (whether handled by judgment, analytic assumptions per a selected design method, or both). Similarly, the distribution of the assigned shear load to the various shear wall segments within a given shear wall line is based on the same principle, but at a different scale. The scale is the subassembly (or shear wall) as opposed to the whole building.

The methods for designing and distributing the forces within a shear wall line differ as described below. As with the three different approaches described for the distribution of lateral building loads, the shear wall design methods place different levels of emphasis on analytic rigor and judgment. Ultimately, the configuration of the building (i.e., are the walls inherently broken into individual segments by large openings or many offsets in plan dimensions?) and the required demand (i.e., shear load) should drive the choice of a shear wall design approach and the resulting construction detailing. Thus, the choice of which design method to use is a matter of designer judgment and required performance. In turn, the design method itself imposes detailing requirements on the final construction in compliance with the analysis assumptions. Accordingly, the above decisions affect the efficiency of the design effort and the complexity of the resulting construction details.

Segmented Shear Wall (SSW) Design Approach

The *segmented shear wall design approach*, well recognized as a standard design practice, is the most widely used method of shear wall design. It considers the shear resisting segments of a given shear wall line as separate “elements,” with each segment restrained against overturning by the use of hold-down connectors at its ends. Each segment is a fully sheathed portion of the wall without any openings for windows or doors. The design shear capacity of each segment is determined by multiplying the length of the segment (sometimes called segment width) by tabulated unit shear design values that are available in the building codes and newer design standards. In its simplest form, the approach analyzes each shear wall segment for static equilibrium in a manner analogous to a cantilevered beam with a fixed end (refer to Figures 6.1 and 6.3a). In a wall with multiple designated shear wall segments, the typical approach to determining an adequate total length of all shear wall segments is to divide the design shear load demand on the wall by the unit shear design value of the wall construction. The effect of stiffness on the actual shear force distribution to the various segments is simply handled by complying with code-required maximum shear wall segment aspect ratios (i.e., segment height divided by segment width). Although an inexact and circuitous method of handling the problem of shear force distribution in a shear wall line, the SSW approach has been in successful practice for many years, partly due to the use of conservative unit shear design values.

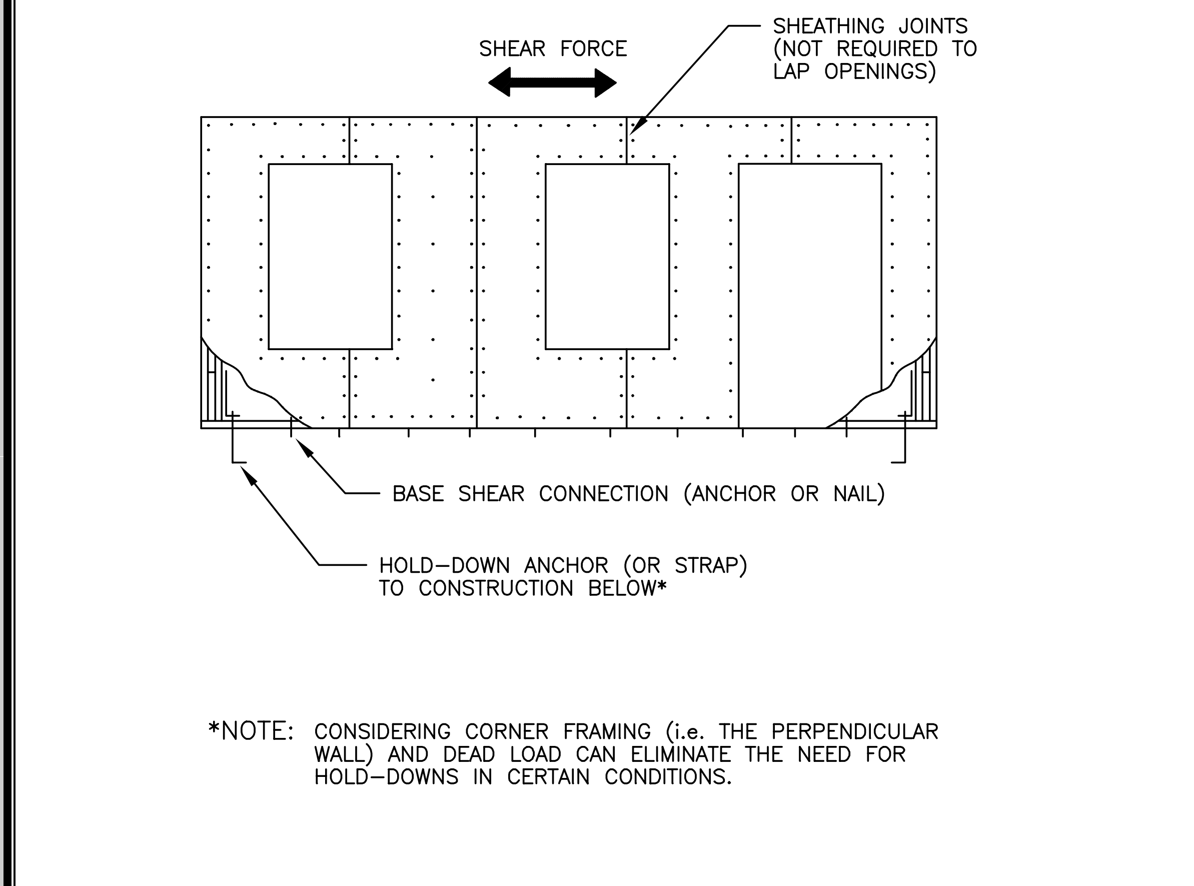
When stiffness is considered, the stiffness of a shear wall segment is assumed to be linearly related to its length (or its total design shear strength). However, the linear relationship is not realistic outside certain limits. For example, stiffness begins to decrease with notable nonlinearly once a shear wall segment decreases below a 4-foot length on an 8-foot-high wall (i.e., aspect ratio of 2 or greater). This does not mean that wall segments shorter than 4 feet in width cannot be used but rather that the effect of relative stiffness in distributing the load needs to be considered. The SSW approach is also less favorable when the wall as a system rather than individual segments (i.e., including sheathed areas above and below openings) may be used to economize on design while meeting required performance (see perforated shear wall design approach below).

As shown in Figure 6.3, it is common either to neglect the contribution of dead load or assume that the dead load on the wall is uniformly distributed as would be the case under gravity loading only. In fact, unless the wall is restrained with an infinitely rigid hold-down device (an impossibility), the uniform dead load distribution will be altered as the wall rotates and deflects upward during the application of shear force (see Figure 6.3b). As a result, depending on the rigidity of the framing system above, the dead load will tend to concentrate more toward the “high points” in the wall line, as the various segments begin to rotate and uplift at their leading edges. Thus, the dead load may be somewhat more effective in offsetting the overturning moment on a shear wall segment than is suggested by the uniform dead load assumption. Unfortunately, this phenomenon involves nonrigid body, nonlinear behavior for which there are no simplified methods of analysis. Therefore, this effect is generally not considered, particularly for walls with specified restraining devices (i.e., hold-downs) that are, by default, generally assumed to be completely rigid–an assumption that is known by testing not to hold true to varying degrees depending on the type of device and its installation.

Basic Perforated Shear Wall (PSW) Design Approach

The *basic perforated shear wall (PSW) design* method is gaining popularity among designers and even earning code recognition. The method, however, is not without controversy in terms of appropriate limits and guidance on use. A perforated shear wall is a wall that is fully sheathed with wood structural panels (i.e., oriented strand board or plywood) and that has openings or “perforations” for windows and doors. The ends of the walls–rather than each individual segment as in the segmented shear wall method–are restrained against overturning. As for the intermediate segments of the wall, they are restrained by conventional or designed framing connections such as those at the base of the wall that transfer the shear force resisted by the wall to the construction below. The capacity of a PSW is determined as the ratio of the strength of a wall with openings to the strength of a wall of the same length without openings. The ratio is calculated by using two empirical equations given in Section 6.5. Figure 6.5 illustrates a perforated shear wall.

FIGURE 6.5 Illustration of a Basic Perforated Shear Wall



The PSW design method requires the least amount of special construction detailing and analysis among the current shear wall design methods. It has been validated in several recent studies in the United States but dates back more than 20 years to research first conducted in Japan (Dolan and Heine, 1997a and b; Dolan and Johnson, 1996a and 1996b; NAHBRC, 1997; NAHBRC, 1998; NAHBRC, 1999; Sugiyama and Matsumoto, 1994; Ni et al., 1998). While it produces the simplest form of an engineered shear wall solution, other methods such as the segmented shear wall design method–all other factors equal–can yield a stronger wall. Conversely, a PSW design with increased sheathing fastening can outperform an SSW with more hold-downs but weaker sheathing fastening. The point is, that for many applications, the PSW method often provides an adequate and more efficient design. Therefore, the PSW method should be considered an option to the SSW method as appropriate.

Enhancements to the PSW Approach

Several options in the form of structural optimizations (i.e., “getting the most from the least”) can enhance the PSW method. One option uses multiple metal straps or ties to restrain each stud, thereby providing a highly redundant and simple method of overturning restraint. Unfortunately, this promising enhancement has been demonstrated in only one known proof test of the concept (NAHBRC, 1999). It can, however, improve shear wall stiffness and increase capacity beyond that achieved with either the basic PSW method or SSW design approach. Another option, subjected to limited study by the NAHB Research Center, calls for perforated shear walls with metal truss plates at key framing joints (NAHBRC, 1998). To a degree similar to that in the first option, this enhancement increases shear capacity and stiffness without the use of any special hold-downs or restraining devices other than conventional framing connections at the base of the wall (i.e., nails or anchor bolts). Neither of the above options applied dead loads to the tested walls, such application would have improved performance. Unfortunately, the results do not lend themselves to easy duplication by analysis and must be used at their face value as empirical evidence to justify practical design improvements for conditions limited by the tests. Analytic methods are under development to facilitate use of optimization concepts in shear wall design and construction.

In a mechanics-based form of the PSW, analytic assumptions using free-body diagrams and principles of statics can conservatively estimate restraining forces that transfer shear around openings in shear walls based on the assumption that wood-framed shear walls behave as rigid bodies with elastic behavior. As compared to several tests of the perforated shear wall method discussed above, the mechanics-based approach leads to a conservative solution requiring strapping around window openings. In a condition outside the limits for application of the PSW method, a mechanics-based design approach for shear transfer around openings provides a reasonable alternative to traditional SSW design and the newer empirically based PSW design. The added detailing merely takes the form of horizontal strapping and blocking at the top and bottom corners of window openings to transfer the calculated forces derived from free-body diagrams representing the shear wall segments and sheathed areas above and below openings. For more detail, the reader should consult other sources of information on this approach (Diekmann, 1986; ICBO, 1997; ICC, 1999).

6.4.3 Basic Diaphragm Design Approach

As described in Chapter 2 and earlier in this section, horizontal diaphragms are designed by using the analogy of a deep beam laid flatwise. Thus, the shear forces in the diaphragm are calculated as for a beam under a uniform load (refer to Figure 6.4). As is similar to the case of shear walls, the design shear capacity of a horizontal diaphragm is determined by multiplying the diaphragm depth (i.e., depth of the analogous deep beam) by the tabulated unit shear design values found in building codes. The chord forces (in the “flange” of the analogous deep beam) are calculated as a tension force and compression force on opposite sides of the diaphragm. The two forces form a force couple (i.e., moment) that resists the bending action of the diaphragm (refer to Figure 6.1).

To simplify the calculation, it is common practice to assume that the chord forces are resisted by a single chord member serving as the “flange” of the deep beam (i.e., a band joist). At the same time, bending forces internal to the diaphragm are assumed to be resisted entirely by the boundary member or band joist rather than by other members and connections within the diaphragm. In addition, other parts of the diaphragm boundary (i.e., walls) that also resist the bending tension and compressive forces are not considered. Certainly, a vast majority of residential roof diaphragms that are not considered "engineered" by current diaphragm design standards have exhibited ample capacity in major design events. Thus, the beam analogy used to develop an analytic model for the design of wood-framed horizontal diaphragms has room for improvement that has yet to be explored from an analytic standpoint.

As with shear walls, openings in the diaphragm affect the diaphragm's capacity. However, no empirical design approach accounts for the effect of openings in a horizontal diaphragm as for shear walls (i.e., the PSW method). Therefore, if openings are present, the effective depth of the diaphragm in resisting shear forces must either discount the depth of the opening or be designed for shear transfer around the opening. If it is necessary to transfer shear forces around a large opening in a diaphragm, it is common to perform a mechanics-based analysis of the shear transfer around the opening. The analysis is similar to the previously described method that uses free-body diagrams for the design of shear walls. The reader is referred to other sources for further study of diaphragm design (Ambrose and Vergun, 1987; APA, 1997; Diekmann, 1986).

6.5 Design Guidelines

6.5.1 General Approach

This section outlines methods for designing shear walls (Section 6.5.2) and diaphragms (Section 6.5.3). The two methods of shear wall design are the segmented shear wall (SSW) method and the perforated shear wall (PSW) method. The selection of a method depends on shear loading demand, wall configuration, and the desired simplicity of the final construction. Regardless of design method and resulting LFRS, the first consideration is the amount of lateral load to be resisted by the arrangement of shear walls and diaphragms in a given building. The design loads and basic load combinations in Chapter 3, Table 3.1, are as follows:

- 0.6D + (W or 0.7E) ASD
- 0.9D + (1.5W or 1.0E) LRFD

Earthquake load and wind load are considered separately, with shear walls designed in accordance with more stringent loading conditions.

Lateral building loads should be distributed to the shear walls on a given story by using one of the following methods as deemed appropriate by the designer:

- tributary area approach;
- total shear approach; or
- relative stiffness approach.

These methods were described earlier (see Section 6.4). In the case of the tributary area method, the loads can be immediately assigned to the various shear wall lines based on tributary building areas (exterior surface area for wind loads and building plan area for seismic loads) for the two orthogonal directions of loading (assuming rectangular-shaped buildings and relatively uniform mass distribution for seismic design). In the case of the total shear approach, the load is considered as a "lump sum" for each story for both orthogonal directions of loading. The shear wall construction and total amount of shear wall for each direction of loading and each shear wall line are then determined in accordance with this section to meet the required load as determined by either the tributary area or total shear approach. The designer must be reasonably confident that the distribution of the shear walls and their resistance is reasonably "balanced" with respect to building geometry and the center of the total resultant shear load on each story. As mentioned, both the tributary and total shear approaches have produced many serviceable designs for typical residential buildings, provided that the designer exercises sound judgment.

In the case of the relative stiffness method, the assignment of loads must be based on an assumed relationship describing the relative stiffness of various shear wall lines. Generally, the stiffness of a wood-framed shear wall is assumed to be directly related to the length of the shear wall segments and the unit shear value of the wall construction. For the perforated shear wall method, the relative stiffness of various perforated shear wall lines may be assumed to be directly related to the design strength of the various perforated shear wall lines. Using the principle of moments and a representation of wall racking stiffness, the designer can then identify the center of shear resistance for each story and determine each story's torsional load (due to the offset of the load center from the center of resistance). Finally, the designer superimposes direct shear loads and torsional shear loads to determine the estimated shear loads on each of the shear wall lines.

It is common practice (and required by some building codes) for the torsional load distribution to be used only to add to the direct shear load on one side of the building but not to subtract from the direct shear load on the other side, even though the restriction is not conceptually accurate. Moreover, most seismic design codes require evaluations of the lateral resistance to seismic loads with "artificial" or "accidental" offsets of the estimated center of mass of the building (i.e., imposition of an "accidental" torsional load imbalance). These provisions, when required, are intended to conservatively address uncertainties in the design process that may otherwise go undetected in any given analysis (i.e., building mass is assumed uniform when it actually is not). As an alternative, uncertainties may be more easily accommodated by increasing the shear load by an equivalent amount in effect (i.e., say 10 percent). Indeed, the seismic shear load using the simplified method (see Equation 3.8-1 in Chapter 3) includes a factor that increases the design load by 20 percent and may be considered adequate to address uncertainties in torsional load distribution. However, the simple "20 percent" approach to addressing accidental torsion loads is not explicitly permitted in any current building code. But, for housing, where many redundancies also exist, the "20 percent" rule seems to be a reasonable substitute for a more "exact" analysis of accidental torsion. Of course, it is not a substitute for evaluating and designing for torsion that is expected to occur.

Design Example 6.5 of Section 6.6 elaborates on and demonstrates the use of the methods of load distribution described above. The reader is encouraged to study and critique them. The example contains many concepts and insights that cannot be otherwise conveyed without the benefit of a “real” problem.

6.5.2 Shear Wall Design

6.5.2.1 Shear Wall Design Values (F_s)

This section provides unfactored (ultimate) unit shear values for wood-framed shear wall constructions that use wood structural panels. Other wall constructions and framing methods are included as an additional resource. The unit shear values given here differ from those in the current codes in that they are based explicitly on the ultimate shear capacity as determined through testing. Therefore, the designer is referred to the applicable building code for "code-approved" unit shear values. This guide uses ultimate unit shear capacities as its basis to give the designer an explicit measure of the actual capacity and safety margin (i.e., reserve strength) used in design and to provide for a more consistent safety margin across various shear wall construction options. Accordingly, it is imperative that the values used in this guide are appropriately adjusted in accordance with Sections 6.5.2.2 and 6.5.2.3 to ensure an acceptable safety margin.

Wood Structural Panels (WSP)

Table 6.1 provides unit shear values for walls sheathed with wood structural panels. It should be noted again that these values are estimates of the ultimate unit shear capacity values as determined from several sources (Tissell, 1993; FEMA, 1997; NAHBRC, 1998; NAHBRC, 1999; others). The design unit shear values in today’s building codes have inconsistent safety margins that typically range from 2.5 to 4 after all applicable adjustments (Tissell, 1993; Soltis, Wolfe, and Tuomi, 1983). Therefore, the actual capacity of a shear wall is not explicitly known to the designer using the codes’ allowable unit shear values. Nonetheless, one alleged benefit of using the code-approved design unit shear values is that the values are believed to address drift implicitly by way of a generally conservative safety margin. Even so, shear wall drift is usually not analyzed in residential construction for reasons stated previously.

The values in Table 6.1 and today’s building codes are based primarily on monotonic tests (i.e., tests that use single-direction loading). Recently, the effect of cyclic loading on wood-framed shear wall capacity has generated considerable controversy. However, cyclic testing is apparently not necessary when determining design values for seismic loading of wood-framed shear walls with structural wood panel sheathing. Depending on the cyclic test protocol, the resulting unit shear values can be above or below those obtained from traditional monotonic shear wall test methods (ASTM, 1998a; ASTM, 1998b). In fact, realistic cyclic testing protocols and their associated interpretations were found to be largely in agreement with the results obtained from monotonic testing (Karacabeyli and Ceccotti, 1998). The differences are generally in the range of 10 percent (plus or minus) and thus seem moot given that the seismic response modifier (see Chapter 3) is based on expert opinion (ATC, 1995) and that the actual performance of light-frame homes does not appear to correlate with important parameters in existing seismic design methods (HUD, 1999), among other factors that currently contribute to design uncertainty.

TABLE 6.1 ***Unfactored (Ultimate) Shear Resistance (plf) for Wood Structural Panel Shear Walls with Framing of Douglas-Fir, Larch, or Southern Pine***[1,2]

			Panels Applied Direct to Framing				
				Nail Spacing at Panel Edges (inches)			
Panel Grade	Nominal Panel Thickness (inches)	Minimum Nail Penetration in Framing (inches) (APA, 1998)	Nail Size (common or galvanized box)	6	4	3	2[3]
Structural I	5/16	1-1/4	6d	821	1,122	1,256	1,333
	3/8[4]	1-3/8	8d	833	1,200	1,362	1,711
	7/16[4]	1-3/8	8d	905	1,356	1,497	1,767
	15/32	1-3/8	8d	977	1,539	1,722	1,800
	15/32	1-1/2	10d[5]	1,256	1,701	1,963	2,222

Notes:
[1]Values are average ultimate unit shear capacity for walls sheathed with Structural I wood structural panels and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3. Additional adjustments to the table values should be made in accordance with those sections. For other rated panels (not Structural I), the table values should be multiplied by 0.85.
[2]All panel edges should be backed with 2-inch nominal or wider framing. Panels may be installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8-inch panels installed with the strong axis parallel to studs spaced 24 inches on-center and 12 inches on-center for other conditions and panel thicknesses.
[3]Framing at adjoining panel edges should be 3-inch nominal or wider and nails should be staggered where nails are spaced 2 inches on-center. A double thickness of nominal 2-inch framing is a suitable substitute.
[4]The values for 3/8- and 7/16-inch panels applied directly to framing may be increased to the values shown for 15/32-inch panels, provided that studs are spaced a maximum of 16 inches on-center or the panel is applied with its strong axis across the studs.
[5]Framing at adjoining panel edges should be 3-inch nominal or wider and nails should be staggered where 10d nails penetrating framing by more than 1-5/8 inches are spaced 3 inches or less on-center. A double thickness of 2-inch nominal framing is a suitable substitute.

The unit shear values in Table 6.1 are based on nailed sheathing connections. The use of elastomeric glue to attach wood structural panel sheathing to wood framing members increases the shear capacity of a shear wall by as much as 50 percent or more (White and Dolan, 1993). Similarly, studies using elastomeric construction adhesive manufactured by 3M Corporation have investigated seismic performance (i.e., cyclic loading) and confirm a stiffness increase of about 65 percent and a shear capacity increase of about 45 to 70 percent over sheathing fastened with nails only (Filiatrault and Foschi, 1991). Rigid adhesives may create even greater strength and stiffness increases. The use of adhesives is beneficial in resisting shear loads from wind. Glued shear wall panels are not recommended for use in high-hazard seismic areas because of the brittle failure mode experienced in the wood framing material (i.e., splitting), though at a significantly increased shear load. Gluing shear wall panels is also not recommended by panel manufacturers because of concern with panel buckling that may occur as a result of the interaction of rigid restraints with moisture/temperature expansion and contraction of the panels.

However, construction adhesives are routinely used in floor diaphragm construction to increase the bending stiffness and strength of floors; in-plane (diaphragm) shear is probably affected by an amount similar to that reported above for shear walls.

For unit shear values of wood structural panels applied to cold-formed steel framing, the following references are suggested: *Uniform Building Code* (ICBO,1997); *Standard Building Code* (SBCCI, 1999); and *Shear Wall Values for Light Weight Steel Framing* (AISI, 1996). The unit shear values for cold-formed steel-framed walls in the previous references are consistent with the values used in Table 6.1, including the recommended safety factor or resistance factor. Table 6.2 presents some typical unit shear values for cold-formed steel-framed walls with wood structural panel sheathing fastened with #8 screws. Values for power-driven, knurled pins (similar to deformed shank nails) should be obtained from the manufacturer and the applicable code evaluation reports (NES, Inc., 1997).

TABLE 6.2 ***Unfactored (Ultimate) Unit Shear Resistance (plf) for Walls with Cold-Formed Steel Framing and Wood Structural Panels***[1,2]

Panel Grade	Panel Type and Nominal Thickness (inches)	Minimum Screw Size[3]	Screw Spacing at Panel Edges (inches)[4]			
			6	4	3	2
Structural I	7/16 OSB	#8	700	915	1,275	1,625
	15/32 plywood	#8	780	990	1,465	1,700

Notes:
[1]Values are average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Values apply to 18 gauge (43 mil) and 20 gage (33 mil) steel C-shaped studs with a 1-5/8-inch flange width and 3-1/2- to 5-1/2-inch depth. Studs spaced a maximum of 24 inches on center.
[3]The #8 screws should have a head diameter of no less than 0.29 inches and the screw threads should penetrate the framing so that the threads are fully engaged in the steel.
[4]The spacing of screws in framing members located in the interior of the panels should be no more than 12 inches on-center.

Portland Cement Stucco (PCS)

Ultimate unit shear values for conventional PCS wall construction range from 490 to 1,580 plf based on the ASTM E 72 test protocol and 12 tests conducted by various testing laboratories (Testing Engineers, Inc., 1971; Testing Engineers, Inc., 1970; ICBO, 1969). In general, nailing the metal lath or wire mesh resulted in ultimate unit shear values less than 750 plf, whereas stapling resulted in ultimate unit shear values greater than 750 plf. An ultimate design value of 500 plf is recommended unless specific details of PCS construction are known. A safety factor of 2 provides a conservative allowable design value of about 250 plf. It must be realized that the actual capacity can be as much as five times 250 plf depending on the method of construction, particularly the means of fastening the stucco lath material. Current code-approved allowable design values are typically about 180 plf (SBCCI, 1999; ICBO, 1997). One code requires the values to be further reduced by 50 percent in higher-hazard seismic design areas (ICBO, 1997), although the reduction factor may not necessarily improve performance with respect to the cracking of the stucco finish in seismic events

(HUD, 1999); refer to Chapter 1 and the discussion in Chapter 3 on displacement compatibility under seismic load. It may be more appropriate to use a lower seismic response modifier R than to increase the safety margin in a manner that is not explicit to the designer. In fact, an R factor for PCS wood-framed walls is not explicitly provided in building codes (perhaps an R of 4.5 for "other" wood-framed walls is used) and should probably be in the range of 3 to 4 (without additional increases in the safety factor) since some ductility is provided by the metal lath and its connection to wood framing.

The above values pertain to PCS that is 7/8-inch thick with nail or staple fasteners spaced 6 inches on-center for attaching the metal wire mesh or lath to all framing members. Nails are typically 11 gauge by 1-1/2 inches in length and staples typically have 3/4-inch leg and 7/8-inch crown dimensions. The above unit shear values also apply to stud spacings no greater than 24 inches on-center. Finally, the aspect ratio of stucco wall segments included in a design shear analysis should not be greater than 2 (height/width) according to current building code practice.

Gypsum Wall Board (GWB)

Ultimate capacities in testing 1/2-inch-thick gypsum wall board range from 140 to 300 plf depending on the fastening schedule (Wolfe, 1983; Patton-Mallory, Gutkowski, Soltis, 1984; NAHBRF, date unknown). Allowable or design unit shear values for gypsum wall board sheathing range from 75 to 150 plf in current building codes depending on the construction and fastener spacing. At least one building code requires the values to be reduced by 50 percent in high-hazard seismic design areas (ICBO, 1997). Gypsum wall board is certainly not recommended as the primary seismic bracing for walls, although it does contribute to the structural resistance of buildings in all seismic and wind conditions. It should also be recognized that fastening of interior gypsum board varies in practice and is generally not an 'inspected" system. Table 6.3 provides estimated ultimate unit shear values for gypsum wall board sheathing.

TABLE 6.3 ***Unfactored (Ultimate) Unit Shear Values (plf) for 1/2-Inch-Thick Gypsum Wall Board Sheathing***[1,2]

GWB Thickness	Blocking Condition[3]	Spacing of Framing (inches)	Fastener Spacing at Pane Edges (inches)				
			12	8	7	6	4
1/2 inch	Blocked	16	120	210	250	260	300
	Unblocked	16	80	170	200	220	250
		24	40	120	150	180	220

Notes:
[1]The values represent average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Fasteners should be minimum 1 1/2-inch drywall nails (i.e., 4d cooler) or 1-1/4-inch drywall screws (i.e., #6 size with bugle head) or equivalent with spacing of fasteners and framing members as shown.
[3]"Blocked" refers to panels with all edges fastened to framing members; "unblocked" refers to the condition where the panels are placed horizontally with horizontal joints between panels not fastened to blocking or vertically with the top and bottom edges fastened only at stud locations.

1x4 Wood Let-in Braces and Metal T-braces

Table 6.4 provides values for typical ultimate shear capacities of 1x4 wood let-in braces and metal T-braces. Though not found in current building codes, the values are based on available test data (Wolfe, 1983; NAHBRF, date unknown). Wood let-in braces and metal T-braces are common in conventional residential construction and add to the shear capacity of walls. They are always used in combination with other wall finish materials that also contribute to a wall's shear capacity. The braces are typically attached to the top and bottom plates of walls and at each intermediate stud intersection with two 8d common nails. They are not recommended for the primary lateral resistance of structures in high-hazard seismic or wind design areas. In particular, values of the seismic response modifier R for walls braced in this manner have not been clearly defined for the sake of standardized seismic design guidance.

TABLE 6.4 ***Unfactored (Ultimate) Shear Resistance (lbs) for 1x4 Wood Let-ins and Metal T-Braces[1,2]***

Type of Diagonal Brace	Ultimate Horizontal Shear Capacity (per brace)[3]
1x4 wood let-in brace (8-foot wall height)[4]	600 lbs (tension and compression)
Metal T-brace[5]	1,400 lbs (tension only)

Notes:
[1]Values are average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Values are based on minimum Spruce-Pine-Fir lumber (specific gravity, G = 0.42).
[3]Capacities are based on tests of wall segments that are restrained against overturning.
[4]Installed with two 8d common nails at each stud and plate intersection. Angle of brace should be between 45 and 60 degrees to horizontal.
[5]Installed per manufacturer recommendations and the applicable code evaluation report. Design values may vary depending on manufacturer recommendations, installation requirements, and product attributes.

Other Shear-Resisting Wall Facings

Just about any wall facing, finish, or siding material contributes to a wall's shear resistance qualities. While the total contribution of nonstructural materials to a typical residential building's lateral resistance is often substantial (i.e., nearly 50 percent if interior partition walls are included), current design codes in the United States prohibit considerations of the role of facing, finish, or siding. Some suggestions call for a simple and conservative 10 percent increase (known as the "whole-building interaction factor") to the calculated shear resistance of the shear walls or a similar adjustment to account for the added resistance and whole-building effects not typically considered in design (Griffiths and Wickens, 1996).

Some other types of wall sheathing materials that provide shear resistance include particle board and fiber board. Ultimate unit shear values for fiber board range from 120 plf (6d nail at 6 inches on panel edges with 3/8-inch panel thickness) to 520 plf (10d nail at 2 inches on panel edges with 5/8-inch panel thickness). The designer should consult the relevant building code or manufacturer data for additional information on fiber board and other materials' shear resistance qualities. In one study that conducted tests on various wall assemblies for HUD, fiber board was not recommended for primary shear resistance in high-hazard seismic or wind design areas for the stated reasons of potential durability and cyclic loading concerns (NAHBRF, date unknown).

Combining Wall Bracing Materials

When wall bracing materials (i.e., sheathing) of the same type are used on opposite faces of a wall, the shear values may be considered additive. In high-hazard seismic design conditions, dissimilar materials are generally assumed to be nonadditive. In wind-loading conditions, dissimilar materials may be considered additive for wood structural panels (exterior) with gypsum wall board (interior). Even though let-in brace or metal T-brace (exterior) with gypsum wall board (interior) and fiber board (exterior) with gypsum wall board (interior) are also additive, they are not explicitly recognized as such in current building codes.

When the shear capacity for walls with different facings is determined in accordance with Sections 6.5.2.2 and 6.5.2.3, the designer must take care to apply the appropriate adjustment factors to determine the wall construction's total design racking strength. Most of the adjustment factors in the following sections apply only to wood structural panel sheathing. Therefore, the adjustments in the next section should be made as appropriate before determining combined shear resistance.

6.5.2.2 Shear Wall Design Capacity

The unfactored and unadjusted ultimate unit shear resistance values of wall assemblies should first be determined in accordance with the guidance provided in the previous section for rated facings or structural sheathing materials used on each side of the wall. This section provides methods for determining and adjusting the design unit shear resistance and the shear capacity of a shear wall by using either the perforated shear wall (PSW) approach or segmented shear wall (SSW) approach discussed in Section 6.4.2. The design approaches and other important considerations are illustrated in the design examples of Section 6.6.

Perforated Shear Wall Design Approach

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

$$F'_s = (F_s)C_{sp}C_{ns}x[\frac{1}{SF} or \phi] \quad \text{(units plf)} \qquad \text{Eq. 6.5-1a}$$

$$F_{psw} = (F'_s)C_{op}C_{dl}x[L] \quad \text{(units lb)} \qquad \text{Eq. 6.5-1b}$$

where,

F_{psw} = the design shear capacity (lb) of the perforated shear wall

F_s = the unfactored (ultimate) and unadjusted unit shear capacity (plf) for each facing of the wall construction; the C_{sp} and C_{ns} adjustment factors apply only to the wood structural panel sheathing F_s values in accordance with Section 6.5.2.1

F'_s = the factored and adjusted design unit shear capacity (plf) for the wall construction

C = the adjustment factors in accordance with Section 6.5.2.3 as applicable

L = the length of the perforated shear wall, which is defined as the distance between the restrained ends of the wall line

1/*SF* = the safety factor adjustment for use with ASD

ϕ = the resistance factor adjustment for use with LRFD

The PSW method (Equations 6.5-1a and b) has the following limits on its use:

- The value of F_s for the wall construction should not exceed 1,500 plf in accordance with Section 6.5.1.2. The wall must be fully sheathed with wood structural panels on at least one side. Unit shear values of sheathing materials may be combined in accordance with Section 6.5.2.1.
- Full-height wall segments within a perforated shear wall should not exceed an aspect ratio of 4 (height/width) unless that portion of the wall is treated as an opening. (Some codes limit the aspect ratio to 2 or 3.5, but recent testing mentioned earlier has demonstrated otherwise.) The first wall segment on either end of a perforated shear wall must not exceed the aspect ratio limitation.
- The ends of the perforated shear wall must be restrained with hold-down devices sized in accordance with Section 6.5.2.4. Hold-down forces that are transferred from the wall above are additive to the hold-down forces in the wall below. Alternatively, each wall stud may be restrained by using a strap sized to resist an uplift force equivalent to the design unit shear resistance F'_s of the wall, provided that the sheathing area ratio r for the wall is not less than 0.5 (see equations for C_{op} and r in Section 6.5.2.3).
- Top plates must be continuous with a minimum connection capacity at splices with lap joints of 1,000 lb, or as required by the design condition, whichever is greater.
- Bottom plate connections to transfer shear to the construction below (i.e., resist slip) should be designed in accordance with Section 6.5.2.5 and should result in a connection at least equivalent to one 1/2-inch anchor bolt at 6 feet on center or two 16d pneumatic nails 0.131-inch diameter at 24 inches on center for wall constructions with $F_sC_{sp}C_{ns}$ not exceeding 800 plf (ultimate capacity of interior and exterior sheathing). Such connections have been shown to provide an ultimate shear slip capacity of more than 800 plf in typical shear wall framing systems (NAHBRC, 1999); refer to Section 7.3.6 of Chapter 7. For wall constructions with ultimate shear capacities $F_sC_{sp}C_{ns}$ exceeding 800 plf, the base connection must be designed to resist the unit shear load and also provide a design uplift resistance equivalent to the design unit shear load.
- Net wind uplift forces from the roof and other tension forces as a result of structural actions above the wall are transferred through

the wall by using an independent load path. Wind uplift may be resisted with the strapping option above, provided that the straps are sized to transfer the additional load.

Segmented Shear Wall Design Approach

The following equations are used to determine the adjusted and factored shear capacity of a shear wall segment:

$$F'_s = F_s C_{sp} C_{ns} C_{ar}[\frac{1}{SF} \text{ or } \phi] \qquad \text{Eq. 6.5-2a}$$

$$F_{ssw} = F'_s \, x[L_s] \qquad \text{Eq. 6.5-2b}$$

where,

F_{ssw} = the design shear capacity (lb) of a single shear wall segment
F_s = the unfactored (ultimate) and unadjusted unit shear resistance (plf) for the wall construction in accordance with Section 6.5.2.1 for each facing of the wall construction; the C_{sp} and C_{ns} adjustment factors apply only to wood structural panel sheathing F_s values
F'_s = the factored (design) and adjusted unit shear resistance (plf) for the total wall construction
C = the adjustment factors in accordance with Section 6.5.2.3
L_s = the length of a shear wall segment (total width of the sheathing panel(s) in the segment)
$1/SF$ = the safety factor adjustment for use with ASD
ϕ = the resistance factor adjustment for use with LRFD

The segmented shear wall design method (Equations 6.5-2a and b) imposes the following limits:

- The aspect ratio of wall segments should not exceed 4 (height/width) as determined by the sheathing dimensions on the wall segment. (Absent an adjustment for the aspect ratio, current codes may restrict the segment aspect ratio to a maximum of 2 or 3.5.)
- The ends of the wall segment should be restrained in accordance with Section 6.5.2.4. Hold-down forces that are transferred from shear wall segments in the wall above are additive to the hold-down forces in the wall below.
- Shear transfer at the base of the wall should be determined in accordance with Section 6.5.2.5.
- Net wind uplift forces from the roof and other tension forces as a result of structural actions above are transferred through the wall by using an independent load path.

For walls with multiple shear wall segments, the design shear resistance for the individual segments may be added to determine the total design shear resistance for the segmented shear wall line. Alternatively, the combined shear capacity at given amounts of drift may be determined by using the load-deformation equations in Section 6.5.2.6.

6.5.2.3 Shear Capacity Adjustment Factors

Safety and Resistance Factors (SF and ϕ)

Table 6.5 recommends values for safety and resistance factors for shear wall design in residential construction. A safety factor of 2.5 is widely recognized for shear wall design, although the range varies substantially in current code-approved unit shear design values for wood-framed walls (i.e., the range is 2 to more than 4). In addition, a safety factor of 2 is commonly used for wind design. The 1.5 safety factor for ancillary buildings is commensurate with lower risk but may not be a recognized practice in current building codes. A safety factor of 2 has been historically applied or recommended for residential dwelling design (HUD, 1967; MPS, 1958; HUD, 1999). It is also more conservative than safety factor adjustments typically used in the design of other properties with wood members and other materials.

TABLE 6.5 ***Minimum Recommended Safety and Resistance Factors for Residential Shear Wall Design***

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

Species Adjustment Factor (C_{sp})

The ultimate unit shear values for wood structural panels in Table 6.1 apply to lumber species with a specific gravity (density), G, greater than or equal to 0.5. Table 6.6 presents specific gravity values for common species of lumber used for wall framing. For G < 0.5, the following value of C_{sp} should be used to adjust values in Table 6.1 only (APA, 1998):

$$C_{sp} = [1 - (0.5 - G)] \leq 1.0 \qquad \text{Eq. 6.5-3}$$

TABLE 6.6 ***Specific Gravity Values (Average) for Common Species of Framing Lumber***

Lumber Species	Specific Gravity, G
Southern Yellow Pine (SYP)	0.55
Douglas Fir-Larch (DF-L)	0.50
Hem-Fir (HF)	0.43
Spruce-Pine-Fir (SPF)	0.42

Nail Size Adjustment Factor (C_{ns})

The ultimate unit shear capacities in Table 6.1 are based on the use of common nails. For other nail types and corresponding nominal sizes, the C_{ns} adjustment factors in Table 6.7 should be used to adjust the values in Table 6.1. Nails should penetrate framing members a minimum of 10D, where D is the diameter of the nail.

TABLE 6.7 Values of C_{ns} for Various Nail Sizes and Types[1]

Nominal Nail Size (penny weight)	Nail Length (inches)	Nail Type		Pneumatic (by diameter in inches)			
		Common[2]	Box[3]	0.092	0.113	0.131	0.148
6d	1-7/8 to 2	1.0	0.8	0.9	1.0	N/A[4]	N/A[4]
8d	2-3/8 to 2-1/2	1.0	0.8	0.5	0.75	1.0	N/A[4]
10d	3	1.0	0.8	N/A[4]	N/A[4]	0.8	1.0

Notes:
[1]The values of C_{ns} are based on ratios of the single shear nail values in NER-272 (NES, Inc., 1997) and the NDS (AF&PA, 1997) and are applicable only to wood structural panel sheathing on wood-framed walls in accordance with Table 6.1.
[2]Common nail diameters are as follows: 6d (0.113 inch), 8d (0.131 inch), and 10d (0.148 inch).
[3]Box nail diameters are as follows: 6d (0.099 inch), 8d (0.113 inch), and 10d (0.128 inch).
[4]Diameter not applicable to nominal nail size. Nail size, diameter, and length should be verified with the manufacturer.

Opening Adjustment Factor (C_{op})

The following equation for C_{op} applies only to the perforated shear wall method in accordance with Equation 6.5-1b of Section 6.5.2.2:

$$C_{op} = r/(3-2r) \qquad \text{Eq. 6.5-4}$$

where,

r = $1/(1 + \alpha/\beta)$ = sheathing area ratio (dimensionless)

α = ΣA_o / (H x L) = ratio of area of all openings ΣA_o to total wall area, H x L (dimensionless)

β = ΣL_i / L = ratio of length of wall with full-height sheathing ΣL_i to the total wall length L of the perforated shear wall (dimensionless)

Dead Load Adjustment Factor (C_{dl})

The C_{dl} factor applies to the perforated shear wall method only (Equation 6.5-1b). The presence of a dead load on a perforated shear has the effect of increasing shear capacity (Ni et al., 1998). The increase is 15 percent for a uniform dead load of 300 plf or more applied to the top of the wall framing. The dead load should be decreased by wind uplift and factored in accordance with the lateral design load combinations of Chapter 3. The C_{dl} adjustment factor is determined as follows and should not exceed 1.15:

$$C_{dl} = 1 + 0.15\left(\frac{w_D}{300}\right) \le 1.15 \qquad \text{Eq 6.5-5}$$

where,

w_D = the net uniform dead load supported at the top of the perforated shear wall (plf) with consideration of wind uplift and factoring in accordance with load combinations of Chapter 3.

Aspect Ratio Adjustment Factor (C_{ar})

The following C_{ar} adjustment factor applies only to the segmented shear wall design method for adjusting the shear resistance of interior and exterior sheathing in accordance with Equation 6.5-2a of Section 6.5.2.2:

$$C_{ar} = \frac{1}{\sqrt{0.5(a)}} \text{ for } 2.0 \le a \le 4.0 \qquad \text{Eq 6.5-6}$$

C_{ar} = 1.0 for $a < 2.0$

where,

a is the aspect ratio (height/width) of the sheathed shear wall segment.

6.5.2.4 Overturning Restraint

Section 6.3 and Figure 6.3 address overturning restraint of shear walls in conceptual terms. In practice, the two generally recognized approaches to providing overturning restraint call for

- the evaluation of equilibrium of forces on a *restrained* shear wall segment using principles of engineering mechanics; or
- the evaluation of *unrestrained* shear walls considering nonuniform dead load distribution at the top of the wall with restraint provided by various connections (i.e., sheathing, wall bottom plate, corner framing, etc.).

The first method applies to restrained shear wall segments in both the perforated and segmented shear wall methods. The first segment on each end of a perforated shear wall is restrained in one direction of loading. Therefore, the overturning forces on that segment are analyzed in the same manner as for a segmented shear wall. The second method listed above is a valid and conceptually realistic method of analyzing the restraint of typical residential wall constructions, but it has not yet fully matured. Further, the method's load path (i.e., distribution of uplift forces to various connections with inelastic properties) is perhaps beyond the practical limits of a designer's intuition. Rather than presume a methodology based on limited testing (see Section 6.3), this guide does not suggest guidelines for the second approach. However, the second method is worth consideration by a designer when attempting to understand the performance of conventional, "nonengineered" residential construction. Mechanics-based methods to assist in the more complicated design approach are under development.

Using basic mechanics as shown in Figure 6.6, the following equation for the chord tension and compression forces are determined by summing moments about the bottom compression or tension side of a restrained shear wall segment:

$$\sum M_C = 0$$

$$F'_s(d)(h) - T(x) - D_W(\tfrac{1}{2}d) - (w_D)(d)(\tfrac{1}{2}d) = 0$$

$$T = \left(\frac{d}{x}\right)\left(F'_s h - \tfrac{1}{2}D_W - \tfrac{1}{2}(w_D)(d)\right) + t \qquad \text{Eq. 6.5-7a}$$

$$\sum M_T = 0$$

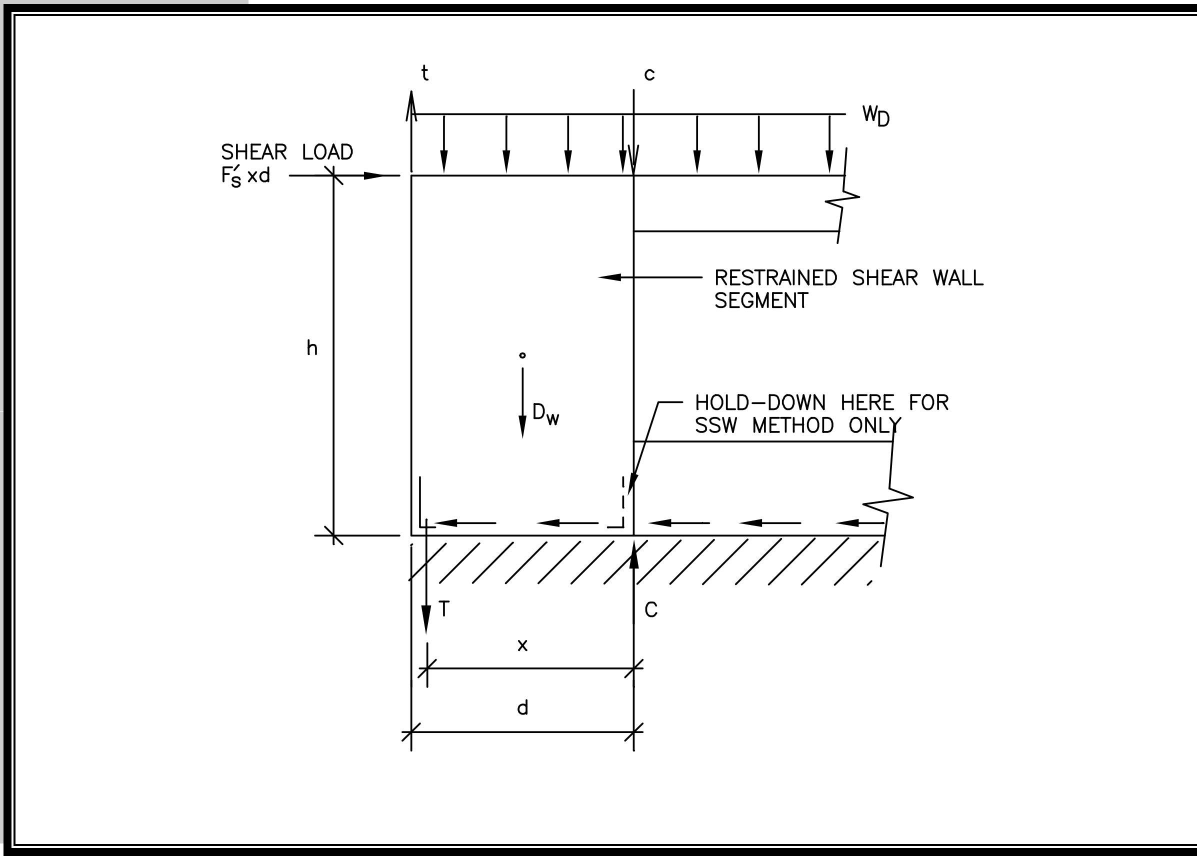
$$C = \left(\frac{d}{x}\right)\left(F'_s h + \tfrac{1}{2}D_W + \tfrac{1}{2}(w_D)(d)\right) + c \qquad \text{Eq. 6.5-7b}$$

where,

- T = the tension force on the hold-down device (lb)
- d = the width of the restrained shear wall segment (ft); for segments greater than 4 ft in width, use d = 4 ft.
- x = the distance between the hold-down device and the compression edge of the restrained shear wall segment (ft); for segments greater than 4 ft in width, use x = 4 ft plus or minus the bracket offset dimension, if any
- F'_s = the design unit shear capacity (plf) determined in accordance with Equation 6.5-2a of Section 6.5.2.2 (for both the PSW and SSW methods)
- h = the height of the wall (ft)
- D_w = the dead load of the shear wall segment (lb); dead load must be factored and wind uplift considered in accordance with the load combinations of Chapter 3.
- w_D = the uniform dead load supported by the shear wall segment (plf); dead load must be factored and wind uplift considered in accordance with the load combinations of Chapter 3.
- t = the tension load transferred through a hold-down device, if any, restraining a wall above (lb); if there is no tension load, t = 0
- c = the compression load transferred from wall segments above, if any (lb); this load may be distributed by horizontal structural elements above the wall (i.e., not a concentrated load); if there is not compression load, c = 0.

The 4-foot-width limit for *d* and *x* is imposed on the analysis of overturning forces as presented above because longer shear wall lengths mean that the contribution of the additional dead load cannot be rigidly transferred through deep bending action of the wall to have a full effect on the uplift forces occurring at the end of the segment, particularly when it is rigidly restrained from uplifting. This effect also depends on the stiffness of the construction above the wall that "delivers" and distributes the load at the top of the wall. The assumptions necessary to include the restraining effects of dead load is no trivial matter and, for that reason, it is common practice to not include any beneficial effect of dead load in the overturning force analysis of individual shear wall segments.

FIGURE 6.6 ***Evaluation of Overturning Forces on a Restrained Shear Wall Segment***



For a more simplified analysis of overturning forces, the effect of dead load may be neglected and the chord forces determined as follows using the symbols defined as before:

$$T = C = \left(\frac{d}{x}\right) F'_s \, h \qquad \text{Eq. 6.5-7c}$$

Any tension or compression force transferred from shear wall overturning forces originating above the wall under consideration must be added to the result of Equation 6.5-7c as appropriate. It is also assumed that any net wind uplift force is resisted by a separate load path (i.e., wind uplift straps are used in addition to overturning or hold-down devices).

For walls not rigidly restrained, the initiation of overturning uplift at the end stud (i.e., chord) shifts an increasing amount of the dead load supported by the wall toward the leading edge. Thus, walls restrained with more flexible hold-down devices or without such devices benefit from increased amounts of offsetting dead load as well as from the ability of wood framing and connections to disperse some of the forces that concentrate in the region of a rigid hold-down device. However, if the bottom plate is rigidly anchored, flexibility in the hold-down device can impose undesirable cross-grain bending forces on the plate due to uplift forces transferred through the sheathing fasteners to the edge of the bottom plate. Further, the sheathing nails in the region of the bottom plate anchor experience greater load and may initiate failure of the wall through an "unzipping" effect.

The proper detailing to balance localized stiffness effects for more even force transfer is obviously a matter of designer judgment. It is mentioned here to emphasize the importance of detailing in wood-framed construction. In particular, wood framing has the innate ability to distribute loads, although weaknesses can develop from seemingly insignificant details. The concern noted above has been attributed to actual problems (i.e., bottom plate splitting) only in severe seismic events and in relatively heavily loaded shear walls. For this reason, it is now common to require larger washers on bottom plate anchor bolts, such as a 2- to 3-inch-square by 1/4-inch-thick plate washer, to prevent the development of cross-grain tension forces in bottom plates in high-hazard seismic regions. The development of high cross-grain tension stresses poses less concern when nails are used to fasten the bottom plate and are located in pairs or staggered on both sides of the wood plate. Thus, the two connection options above represent different approaches. The first, using the plate washers, maintains a rigid connection throughout the wall to prevent cross grain tension in the bottom plate. The second, using nails, is a more "flexible" connection that prevents concentrated cross-grain bending forces from developing. With sufficient capacity provided, the nailing approach may yield a more "ductile" system. Unfortunately, these intricate detailing issues are not accommodated in the single seismic response modifier used for wood-framed shear walls or the provisions of any existing code. These aspects of design are not easily "quantified" and are considered matters of qualitative engineering judgment.

Finally, it is important to recognize that the hold-down must be attached to a vertical wall framing member (i.e., a stud) that receives the wood structural panel edge nailing. If not, the hold-down will not be fully effective (i.e., the overturning forces must be "delivered" to the hold-down through the sheathing panel edge nailing). In addition, the method of deriving hold-down capacity ratings may vary from bracket to bracket and manufacturer to manufacturer. For some brackets, the rated capacity may be based on tests of the bracket itself that do not represent its use in an assembly (i.e., as attached to a wood member). Many hold-down brackets transfer tension through an eccentric load path that creates an end moment on the vertical framing member to which it is attached. Therefore, there may be several design considerations in specifying an appropriate hold-down device that go beyond simply selecting a device with a sufficient rated capacity from manufacturer literature. In response to these issues, some local codes may require certain reductions to or verification of rated hold-down capacities.

6.5.2.5 Shear Transfer (Sliding)

The sliding shear at the base of a shear wall is equivalent to the shear load input to the wall. To ensure that the sliding shear force transfer is balanced with the shear capacity of the wall, the connections at the base of the wall are usually designed to transfer the design unit shear capacity F'_s of the shear wall. Generally, the connections used to resist sliding shear include anchor bolts (fastening to concrete) and nails (fastening to wood framing). Metal plate connectors may also be used (consult manufacturer literature). In what is a conservative decision, frictional resistance and "pinching" effects usually go ignored. However, if friction is considered, a friction coefficient of 0.3 may be multiplied by the dead load normal to the slippage plane to determine a nominal resistance provided by friction.

As a modification to the above rule, if the bottom plate is continuous in a perforated shear wall, the sliding shear resistance is the capacity of the perforated shear wall F_{psw}. If the bottom plate is not continuous, then the sliding shear should be designed to resist the design unit shear capacity of the wall construction F'_s as discussed above. Similarly, if the restrained shear wall segments in a segmented shear wall line are connected to a continuous bottom plate extending between shear wall segments, then the sliding shear can be distributed along the entire length of the bottom plate. For example, if two 4-foot shear wall segments are located in a wall 12 feet long with a continuous bottom plate, then the unit sliding shear resistance required at the bottom plate anchorage is (8 ft)(F'_s)/(12 ft) or 2/3(F'_s). This is similar to the mechanism by which a unit shear load is transferred from a horizontal diaphragm to the wall top plate and then into the shear wall segments through a collector (i.e., top plate). Chapter 7 addresses design of the above types of shear connections.

6.5.2.6 Shear Wall Stiffness and Drift

The methods for predicting shear wall stiffness or drift in this section are based on idealized conditions representative solely of the testing conditions to which the equations are related. The conditions do not account for the many factors that may decrease the actual drift of a shear wall in its final construction. As mentioned, shear wall drift is generally overestimated in comparison with actual behavior in a completed structure (see Section 6.2 on whole-building tests). The degree of overprediction may reach a factor of 2 at design load conditions. At capacity, the error may not be as large because some nonstructural components may be past their yield point.

At the same time, drift analysis may not consider the factors that also increase drift, such as deformation characteristics of the hold-down hardware (for hardware that is less stiff than that typically used in testing), lumber shrinkage (i.e., causing time-delayed slack in joints), lumber compression under heavy shear wall compression chord load, and construction tolerances. Therefore, the results of a drift analysis should be considered as a guide to engineering judgment, not an exact prediction of drift.

The load-drift equations in this section may be solved to yield shear wall resistance for a given amount of shear wall drift. In this manner, a series of shear wall segments or even perforated shear walls embedded within a given wall line may be combined to determine an overall load-drift relationship for the entire wall line. The load-drift relationships are based on the nonlinear behavior of wood-framed shear walls and provide a reasonably accurate means of determining the behavior of walls of various configurations. The relationship may also be used for determining the relative stiffness of shear wall lines in conjunction with the relative stiffness method of distributing lateral building loads and for considering torsional behavior of a building with a nonsymmetrical shear wall layout in stiffness and in geometry. The approach is fairly straightforward and is left to the reader for experimentation.

Perforated Shear Wall Load-Drift Relationship

The load-drift equation below is based on several perforated shear wall tests already discussed in this chapter. It provides a nonlinear load-drift relationship up to the ultimate capacity of the perforated shear wall as determined in Section 6.5.2.2. When considering shear wall load-drift behavior in an actual building, the reader is reminded of the aforementioned accuracy issues; however, accuracy relative to the test data is reasonable (i.e., plus or minus 1/2-inch at capacity).

$$\Delta = 1.8\left(\frac{0.5}{G}\right)\left(\frac{1}{\sqrt{r}}\right)\left(\frac{V_d}{F_{PSW,ULT}}\right)^{2.8}\left[\frac{h}{8}\right] \text{ (inches)} \qquad \text{Eq. 6.5-8}$$

where,

Δ = the shear wall drift (in) at shear load demand, V_d (lb)
G = the specific gravity of framing lumber (see Table 6.6)
r = the sheathing area ratio (see Section 6.5.2.3, C_{op})
V_d = the shear load demand (lb) on the perforated shear wall; the value of V_d is set at any unit shear demand less than or equal to $F_{psw,ult}$ while the value of V_d should be set to the design shear load when checking drift at design load conditions
$F_{psw,ult}$ = the unfactored (ultimate) shear capacity (lb) for the perforated shear wall (i.e., F_{psw} x *SF* or F_{psw}/ϕ for ASD and LRFD, respectively)
h = the height of wall (ft)

Segmented Shear Wall Load-Drift Relationship

APA Semiempirical Load-Drift Equation

Several codes and industry design guidelines specify a deflection equation for shear walls that includes a multipart estimate of various factors' contribution to shear wall deflection (ICBO, 1997; ICC, 1999, APA, 1997). The approach relies on a mix of mechanics-based principles and empirical modifications. The principles and modifications are not repeated here because the APA method of drift prediction is considered no more reliable than that presented next. In addition, the equation is complex relative to the ability to predict drift accurately. It also requires adjustment factors, such as a nail-slip factor, that can only be determined by testing.

Empirical, Nonlinear Load-Drift Equation

Drift in a wood structural panel shear wall segment may be approximated in accordance with the following equation:

$$\Delta = 2.2\left(\frac{0.5}{G}\right)\sqrt[4]{a}\left(\frac{V_d}{F_{SSW,ULT}}\right)^{2.8}\left[\frac{h}{8}\right] \text{ (in)} \qquad \text{Eq. 6.5-9}$$

where,

Δ = the shear wall drift (in) at load V_d (lb)
G = the specific gravity of framing lumber
a = the shear wall segment aspect ratio (height/width) for aspect ratios from 4 to 1; a value of 1 shall be used for shear wall segments with width (length) greater than height
V_d = the shear load demand (lb) on the wall; the value of V_d is set at any unit shear demand less than or equal to $F_{ssw,ult}$ while the value of V_d should be set to the design load when checking drift at design load conditions
$F_{ssw,ult}$ = the unfactored (ultimate) shear capacity (lb) of the shear wall segment (i.e., F_{ssw} x *SF* or F_{ssw}/ϕ for ASD and LRFD, respectively)
h = the height of wall (ft)

The above equation is based on several tests of shear wall segments with aspect ratios ranging from 4:1 to 1:5.

6.5.2.7 Portal Frames

In situations with little space to include sufficient shear walls to meet required loading conditions, the designer must turn to alternatives. An example is a garage opening supporting a two-story home on a narrow lot such that other wall openings for windows and an entrance door leaves little room for shear walls. One option is to consider torsion and the distribution of lateral loads in accordance with the relative stiffness method. Another possibility is the use of a portal frame.

Portal frames may be simple, specialized framing details that can be assembled on site. They use fastening details, metal connector hardware, and sheathing to form a wooden moment frame and, in many cases, perform adequately. Various configurations of portal frames have undergone testing and provide data and details on which the designer can base a design (NAHBRC, 1998; APA, 1994). The ultimate shear capacity of portal frames ranges from 2,400 to more than 6,000 pounds depending on the complexity and strength of the construction details. A simple detail involves extending a garage header so that it is end-nailed to a full-height corner stud, strapping the header to the jamb studs at the portal opening, attaching sheathing with a standard nailing schedule, and anchoring the portal frame with typical perforated shear wall requirements. The system has an ultimate shear capacity of about 3,400 pounds that, with a safety factor of 2 to 2.5, provides a simple solution for many portal frame applications for residential construction in high-hazard seismic or wind regions. Several manufacturers offer preengineered portal frame and shear wall elements that can be ordered to custom requirements or standard conditions.

6.5.3 Diaphragm Design

6.5.3.1 Diaphragm Design Values

Depending on the location and number of supporting shear wall lines, the shear and moments on a diaphragm are determined by using the analogy of a simply supported or continuous span beam. The designer uses the shear load on the diaphragm per unit width of the diaphragm (i.e., floor or roof) to select a combination of sheathing and fastening from a table of allowable horizontal diaphragm unit shear values found in U.S. building codes. Similar to those for shear walls, unit shear values for diaphragms vary according to sheathing thickness and nailing schedules, among other factors. Table 6.8 presents several of the more common floor and roof constructions used in residential construction as well as their allowable diaphragm resistance values. The values include a safety factor for ASD and therefore require no additional factoring. The aspect ratio of a diaphragm should be no greater than 4 (length/width) in accordance with current building code limits. In addition, the sheathing attachment in floor diaphragms is often supplemented with glue or construction adhesive. The increase in unit shear capacity of vertical diaphragms (i.e. shear walls) was discussed in Section 6.5.2.1 in association with Table 6.1. A similar increase to the unit shear capacity of floor diaphragms can be expected, not to mention increased stiffness when the floor sheathing is glued and nailed.

TABLE 6.8 ***Horizontal Diaphragm ASD Shear Values (plf) for Unblocked Roof and Floor Construction Using Douglas Fir or Southern Pine Framing***[1,2,3,4]

Panel Type and Application	Nominal Panel Thickness (inches)	Common Nail Size	Design Shear Value (plf)
Structural I (Roof)	5/16	6d	165
	3/8	8d	185
	15/32	10d	285
APA Sturd-I-Floor (Floor) and Rated Sheathing	7/16	8d	230
	15/32	8d	240
	19/32	10d	285

Notes:
[1]Minimum framing member thickness is 1-1/2 inches.
[2]Nails spaced at 6 inches on-center at supported panel edges and at the perimeter of the diaphragm. Nails spaced at 12 inches on-center on other framing members spaced a maximum of 24 inches on-center.
[3]"Unblocked" means that sheathing joints perpendicular to framing members are not fastened to blocking.
[4]Apply C_{sp} and C_{ns} adjustment factors to table values as appropriate (see Section 6.5.2.3 for adjustment factor values).

6.5.3.2 Diaphragm Design

As noted, diaphragms are designed in accordance with simple beam equations. To determine the shear load on a simply supported diaphragm (i.e., diaphragm supported by shear walls at each side), the designer uses the following equation to calculate the unit shear force to be resisted by the diaphragm sheathing:

$$V_{max} = \frac{1}{2} wl \quad \text{Eq. 6.5-10a}$$

$$v_{max} = \frac{V_{max}}{d} \quad \text{Eq. 6.5-10b}$$

where,

V_{max} = the maximum shear load on the diaphragm (plf)
w = the tributary uniform load (plf) applied to the diaphragm resulting from seismic or wind loading
l = the length of the diaphragm perpendicular to the direction of the load (ft)
v_{max} = the unit shear across the diaphragm in the direction of the load (plf)
d = the depth or width of the diaphragm in the direction of the load (ft)

The following equations are used to determine the theoretical chord tension and compression forces on a simply supported diaphragm as described above:

$$M_{max} = \frac{1}{8} wl^2 \quad \text{Eq. 6.5-11a}$$

$$T_{max} = C_{max} = \frac{M_{max}}{d} \quad \text{Eq. 6.5-11b}$$

where,

M_{max} = the bending moment on the diaphragm (ft-lb)
w = the tributary uniform load (plf) applied to the diaphragm resulting from seismic or wind loading
l = the length of the diaphragm perpendicular to the direction of the load (ft)
T_{max} = the maximum chord tension force (lb)
C_{max} = the maximum chord compression force (lb)
d = the depth or width of the diaphragm in the direction of the load (ft)

If the diaphragm is not simply supported at its ends, the designer uses appropriate beam equations (see Appendix A) in a manner similar to that above to determine the shear and moment on the diaphragm. The calculations to determine the unit shear in the diaphragm and the tension and compression in the chords are also similar to those given above. It should be noted that the maximum chord forces occur at the location of the maximum moment. For a simply supported diaphragm, the maximum chord forces occur at mid-span between the perimeter shear walls. Thus, chord requirements may vary depending on location and magnitude of the bending moment on the diaphragm. Similarly, shear forces on a simply supported diaphragm are highest near the perimeter shear walls (i.e., reactions). Therefore, nailing requirements for diaphragms may be adjusted depending on the variation of the shear force in interior regions of the diaphragm. Generally, these variations are not critical in small residential structures such that fastening schedules can remain constant throughout the entire diaphragm. If there are openings in the horizontal diaphragm, the width of the opening dimension is usually discounted from the width *d* of the diaphragm when determining the unit shear load on the diaphragm.

6.5.3.3 Shear Transfer (Sliding)

The shear forces in the diaphragm must be adequately transferred to the supporting shear walls. For typical residential roof diaphragms, conventional roof framing connections are often sufficient to transfer the small sliding shear forces to the shear walls (unless heavy roof coverings are used in high-hazard seismic areas or steep roof slopes are used in high-hazard wind regions). The transfer of shear forces from floor diaphragms to shear walls may also be handled by conventional nailed connections between the floor boundary member (i.e., a band joist or end joist that is attached to the floor diaphragm sheathing) and the wall framing below. In heavily loaded conditions, metal shear plates may supplement the connections. The simple rule to follow for these connections is that the shear force in from the diaphragm must equal the shear force out to the supporting wall. Floors supported on a foundation wall are usually connected to a wood sill plate bolted to the foundation wall; however, the floor joist and/or the band joist may be directly connected to the foundation wall. Chapter 7 addresses the design of these shear connections.

6.5.3.4 Diaphragm Stiffness

Diaphragm stiffness may be calculated by using semi-empirical methods based on principles of mechanics. The equations are found in most modern building codes and industry guidelines (APA, 1997; ICBO, 1997; ICC, 1999). For typical residential construction, however, the calculation of diaphragm deflection is almost never necessary and rarely performed. Therefore, the equations and their empirical adjustment factors are not repeated here. Nonetheless, the designer who attempts diaphragm deflection or stiffness calculations is cautioned regarding the same accuracy concerns mentioned for shear wall drift calculations. The stiffness of floor and roof diaphragms is highly dependent on the final construction, including interior finishes (see Section 6.2 on whole-building tests).

6.6 Design Examples

EXAMPLE 6.1 *Segmented Shear Wall Design*

Given

The segmented shear wall line, as shown in the figure below, has the following dimensions:

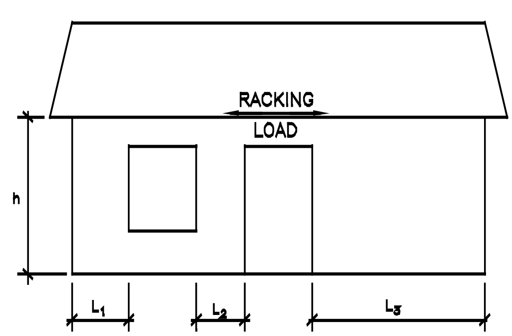
h = 8 ft
L_1 = 3 ft
L_2 = 2 ft
L_3 = 8 ft

Wall construction:

- Exterior sheathing is 7/16-inch-thick OSB with 8d pneumatic nails (0.113 inch diameter by 2 3/8 inches long) spaced 6 inches on center on panel edges and 12 inches on center in panel field
- Interior sheathing is 1/2-inch-thick gypsum wall board with #6 screws at 12 inches on center
- Framing lumber is Spruce-Pine-Fir, Stud grade (specific gravity, G = 0.42); studs are spaced at 16 inches on center.

Loading condition (assumed for illustration)

Wind shear load on wall line = 3,000 lb
Seismic shear load on wall line = 1,000 lb



Find

1. Design capacity of the segmented shear wall line for wind and seismic shear resistance.
2. Base shear connection requirements.
3. Chord tension and compression forces.
4. Load-drift behavior of the segmented shear wall line and estimated drift at design load conditions.

Solution

1. Determine the factored and adjusted (design) shear capacities for the wall segments and the total wall line (Section 6.5.2).

$F_{s,ext}$ = 905 plf OSB sheathing (Table 6.1)
$F_{s,int}$ = 80 plf GWB sheathing (Table 6.3)

The design shear capacity of the wall construction is determined as follows for each segment (Sections 6.5.2.1 and 6.5.2.2):

$F'_s = F'_{s,ext} + F'_{s,int}$
$F'_s = F_{s,ext}\, C_{sp}\, C_{ns}\, C_{ar}\, [1/SF] + F_{s,int}\, C_{ar}\, [1/SF]$

C_{sp} = [1-(0.5-0.42)] = 0.92 (Section 6.5.2.3)
C_{ns} = 0.75 (Table 6.7)
SF = 2.0 (wind) or 2.5 (seismic) (Table 6.5)

Segment 1

a = h/L_1 = (8 ft)/(3 ft) = 2.67 (segment aspect ratio)
C_{ar} = 1/sqrt(0.5(a)) = 0.87 (Section 6.5.2.3)

For wind design

$F'_{s,1,wind}$ = (905 plf)(0.92)(0.75)(0.87)(1/2.0) + (80 plf)(0.87)(1/2.0)
= 272 plf + 35 plf = 307 plf
$F_{ssw,1,wind}$ = $F'_s(L_1)$ = (307 plf)(3 ft) = 921 lb

For seismic design

$F'_{s,1,seismic}$ = (905 plf)(0.92)(0.75)(0.87)(1/2.5) + 0 = 218 plf
$F_{ssw,1,seismic}$ = (218 plf)(3 ft) = 654 lb

Segment 2

a = h/L_2 = (8 ft)/(2 ft) = 4
C_{ar} = 1/sqrt(0.5(a)) = 0.71

For wind design

$F'_{s,2,wind}$ = (905 plf)(0.92)(0.75)(0.71)(1/2.0) + (80 plf)(0.71)(1/2.0)
= 222 plf + 28 plf = 250 plf
$F_{ssw,2,wind}$ = (250 plf)(2 ft) = 500 lb

For seismic design

$F'_{s,2,seismic}$ = (905 plf)(0.92)(0.75)(0.71)(1/2.5) + 0 = 178 plf
$F_{ssw,2,seismic}$ = (178 plf)(2 ft) = 356 lb

Segment 3

$a = h/L_3 = (8\text{ ft})/(8\text{ ft}) = 1$
$C_{ar} = 1.0$ (for $a < 2$)

For wind design

$F'_{s,3,wind} = (905\text{ plf})(0.92)(0.75)(1.0)(1/2.0) + (80\text{ plf})(1.0)(1/2.0)$
$= 312\text{ plf} + 40\text{ plf} = 352\text{ plf}$
$F_{ssw,3,wind} = (352\text{ plf})(8\text{ ft}) = 2{,}816\text{ lb}$

For seismic design

$F'_{s,3,seismic} = (905\text{ plf})(0.92)(0.75)(1.0)(1/2.5) + 0 = 250\text{ plf}$
$F_{ssw,3,seismic} = (250\text{ plf})(8\text{ ft}) = 2{,}000\text{ lb}$

Total for wall line

$F_{ssw,total,wind} = 921\text{ lb} + 500\text{ lb} + 2{,}816\text{ lb} = 4{,}237\text{ lb}$
$F_{ssw,total,seismic} = 654\text{ lb} + 356\text{ lb} + 2{,}000\text{ lb} = 3{,}010\text{ lb}$

2. Determine base shear connection requirements to transfer shear load to the foundation or floor construction below the wall

The wall bottom plate to the left of the door opening is considered to be continuous and therefore acts as a distributor of the shear load resisted by Segments 1 and 2. The uniform shear connection load on the bottom plate to the left of the opening is determined as follows:

Bottom plate length $= 3\text{ ft} + 3\text{ ft} + 2\text{ ft} = 8\text{ ft}$

Base shear resistance required (wind) $= (F_{ssw,1,wind} + F_{ssw,2,wind})/(\text{plate length})$
$= (921\text{ lb} + 500\text{ lb})/(8\text{ ft}) = 178\text{ plf}$

Base shear resistance required (seismic) $= (F_{ssw,1,seismic} + F_{ssw,2,seismic})/(\text{plate length})$
$= (654\text{ lb} + 356\text{ lb})/(8\text{ ft}) = 127\text{ plf}$

For the wall bottom plate to the right of the door opening, the base shear connection is equivalent to $F'_{s,3,wind} = 352$ plf or $F'_{s,3,seismic} = 250$ plf for wind and seismic design respectively.

Normally, this connection is achieved by use of nailed or bolted bottom plate fastenings. Refer to Chapter 7 and Section 7.3.6 for information on designing these connections.

Notes:

1. While the above example shows that variable bottom plate connections may be specified based on differing shear transfer requirements for portions of the wall, it is acceptable practice to use a constant (i.e., worst-case) base shear connection to simplify construction. However, this can result in excessive fastening requirements for certain loading conditions and shear wall configurations.
2. For the assumed wind loading of 3,000 lb, the wall has excess design capacity (i.e., 4,237 lb). The design wind load may be distributed to the shear wall segments in proportion to their design capacity (as shown in the next step for hold-down design) to reduce the shear connection loads accordingly. For seismic design, this should not be done and the base shear connection design should be based on the design capacity of the shear walls to ensure that a "balanced design" is achieved (i.e., the base connection capacity meets or exceeds that of the shear wall). This approach is necessary in seismic design because the actual shear force realized in the connections may be substantially higher than anticipated by the design seismic load calculated using an R factor in accordance with Equation 3.8-1 of Chapter 3. Refer also to the discussion on R factors and overstrength in Section 3.8.4 of Chapter 3. It should be realized that the GWB interior finish design shear capacity was not included in determining the design shear wall capacity for seismic loading. While this is representative of current building code practice, it can create a situation where the actual shear wall capacity and connection forces experienced are higher than those used for design purposes. This condition (i.e., underestimating of the design shear wall capacity) should also be considered in providing sufficiently strong overturning connections (i.e., hold-downs) as covered in the next step.

3. Determine the chord tension and compression (i.e., overturning) forces in the shear wall segments (Section 6.5.2.4)

Basic equation for overturning (Equation 6.5-7c)

$T = C = (d/x)(F'_s)(h)$

<u>Segment 1</u>

h = 8 ft
d = 3 ft
x = d – (width of end studs + offset to center of hold-down anchor bolt)*
= 3 ft – (4.5 in + 1.5 in)(1ft/12 in) = 2.5 ft

*If an anchor strap is used, the offset dimension may be reduced from that determined above assuming a side-mounted hold-down bracket. Also, depending on the number of studs at the end of the wall segment and the type of bracket used, the offset dimension will vary and must be verified by the designer.

$F'_{s,1,wind}$ = 307 plf
$F'_{s,1,seismic}$ = 218 plf

T = C = (3 ft / 2.5 ft)(307 plf)(8 ft) = 2,947 lb (wind)
T = C = (3 ft / 2.5 ft)(218 plf)(8 ft) = 2,093 lb (seismic)

Segment 2

h = 8 ft
d = 2 ft
x = 2 ft – 0.5 ft = 1.5 ft
$F'_{s,2,wind}$ = 250 plf
$F'_{s,2,seismic}$ = 178 plf

T = C = (2 ft / 1.5 ft)(250 plf)(8 ft) = 2,667 lb (wind)
T = C = (2 ft / 1.5 ft)(178 plf)(8 ft) = 1,899 lb (seismic)

Segment 3

h = 8 ft
d = 8 ft
x = 8 ft – 0.5 ft = 7.5 ft
$F'_{s,2,wind}$ = 352 plf
$F'_{s,2,seismic}$ = 250 plf

T = C = (8 ft / 7.5 ft)(352 plf)(8 ft) = 3,004 lb (wind)
T = C = (8 ft / 7.5 ft)(250 plf)(8 ft) = 2,133 lb (seismic)

Notes:

1. In each of the above cases, the seismic tension and compression forces on the shear wall chords are less than that determined for the wind loading condition. This occurrence is the result of using a larger safety factor to determine the shear wall design capacity and the practice of not including the interior sheathing (GWB) design shear capacity for seismic design. Thus, the chord forces based on the seismic shear wall design capacity may be under-designed unless a sufficient safety factor is used in the manufacturer's rated hold-down capacity to compensate. In other words, the ultimate capacity of the hold-down connector should be greater than the overturning force that could be created based on the ultimate shear capacity of the wall, including the contribution of the interior GWB finish. This condition should be verified by the designer since the current code practice may not provide explicit guidance on the issue of balanced design on the basis of system capacity (i.e., connector capacity relative to shear wall capacity). This issue is primarily a concern with seismic design because of the higher safety factor used to determine design shear wall capacity and the code practice not to include the contributing shear capacity of the interior finish.
2. The compression chord force should be recognized as not being a point load at the top of the stud(s) comprising the compression chord. Rather, the compression chord force is accumulated through the sheathing and begins at the top of the wall with a value of zero and increases to C (as determined above) at the base of the compression chord. Therefore, this condition will affect how the compression chord is modeled from the standpoint of determining its capacity as a column using the column equations in the NDS.
3. The design of base shear connections and overturning forces assume that the wind uplift forces at the base of the wall are offset by 0.6 times the dead load (ASD) at that point in the load path or that an additional load path for uplift is provided by metal strapping or other means.
4. As mentioned in Step 2 for the design of base shear connections, the wind load on the designated shear wall segments may be distributed according to the design capacity of each segment in proportion to that of the total shear wall line. This method is particularly useful when the design shear capacity of the wall line is substantially higher than the shear demand required by the wind load as is applicable to this hypothetical example. Alternatively, a shear wall segment may be eliminated from the analysis by not specifying restraining devices for the segment (i.e., hold-down brackets). If the former approach is taken, the wind load is distributed as follows:

Fraction of design wind load to Segment 1:

$F_{ssw,1,wind} / F_{ssw,total,wind} = (921\ lb)/(4{,}237\ lb) = 0.22$

Fraction of wind load to Segment 2:

$F_{ssw,2,wind} / F_{ssw,total,wind} = (500\ lb)/(4{,}237\ lb) = 0.12$

Fraction of wind load to Segment 3:

$F_{ssw,3,wind} / F_{ssw,total,wind} = (2{,}816\ lb)/(4{,}237\ lb) = 0.66$

Thus, the unit shear load on each shear wall segment due to the design wind shear of 3,000 lb on the total wall line is determined as follows:

Segment 1:	0.22(3,000 lb)/(3 ft) = 220 plf
Segment 2:	0.12(3,000 lb)/(2 ft) = 180 plf
Segment 3:	0.66(3,000 lb)/(8 ft) = 248 plf

Now, the overturning forces (chord forces) determined above and the base shear connection requirements determined in Step 2 may be recalculated by substituting the above values, which are based on the design wind loading. This approach only applies to the wind loading condition when the design wind loading on the wall line is less than the design capacity of the wall line. As mentioned, it may be more efficient to eliminate a designed shear wall segment to bring the total design shear capacity more in line with the design wind shear load on the wall. Alternatively, a lower capacity shear wall construction may be specified to better match the loading condition (i.e., use a thinner wood structural sheathing panel, etc.). This decision will depend on the conditions experienced in other walls of the building such that a single wall construction type may be used throughout for all exterior walls (i.e., simplified construction).

4. Determine the load-drift behavior of the wall line.

Only the load-drift behavior for wind design is shown below. For seismic design, a simple substitution of the design shear capacities of the wall segments and the safety factor for seismic design (as determined previously) may be used to determine a load-drift relationship for use in seismic design.

The basic equation for load-drift estimation of a shear wall segment is as follows:

$$\Delta = 2.2\left(\frac{0.5}{G}\right)\sqrt[4]{a}\left(\frac{V_d}{F_{SSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \qquad \text{(Equation 6.5-9)}$$

h = 8 ft

G = 0.42 (Spruce-Pine-Fir)

Aspect ratios for the wall segments

$a_1 = 2.67$

$a_2 = 4.0$

$a_3 = 1.0$

$F_{ssw,ult,1,wind} = F_{ssw,1,wind}(SF) = (921\ lb)(2.0) = 1{,}842\ lb$

$F_{ssw,ult,2,wind} = F_{ssw,2,wind}(SF) = (500\ lb)(2.0) = 1{,}000\ lb$

$F_{ssw,ult,3,wind} = F_{ssw,3,wind}(SF) = (2{,}816\ lb)(2.0) = 5{,}632\ lb$

Therefore, the total ultimate capacity of the wall for wind loading is

$F_{ssw,ult,wall,wind}$ = 1,842 lb + 1,000 lb + 5,632 lb = 8,474 lb

Substituting the above values into the basic load-drift equation above, the following load-drift equations are determined for each segment:

Segment 1: $\Delta_1 = 2.41\text{x}10^{-9} (V_{d,1,wind})^{2.8}$ (inches)
Segment 2: $\Delta_2 = 1.45\text{x}10^{-8} (V_{d,2,wind})^{2.8}$ (inches)
Segment 1: $\Delta_3 = 2.41\text{x}10^{-10} (V_{d,3,wind})^{2.8}$ (inches)

Realizing that each segment must deflect equally (or nearly so) as the wall line deflects, the above deflections may be set equivalent to the total wall line drift as follows:

$\Delta_{wall} = \Delta_1 = \Delta_2 = \Delta_3$

Further, the above equations may be solved for V_d as follows:

Segment 1: $V_{d,1,wind} = 1{,}196 (\Delta_{wall})^{0.36}$
Segment 2: $V_{d,2,wind} = 630 (\Delta_{wall})^{0.36}$
Segment 3: $V_{d,3,wind} = 1{,}997 (\Delta_{wall})^{0.36}$

The sum of the above equations must equal the wind shear load (demand) on the wall at any given drift of the wall as follows:

$V_{d,wall,wind} = V_{d,1,wind} + V_{d,2,wind} + V_{d,3,wind} = 3{,}823 (\Delta_{wall})^{0.36}$

Solving for Δ_{wall} , the following final equation is obtained for the purpose of estimating drift and any given wind shear load from zero to $F_{ssw,ult,wall,wind}$:

$\Delta_{wall} = 9.32\text{x}10^{-11}(V_{d,wall,wind})^{2.8}$

For the design wind load on the wall of 3,000 lb as assumed in this example, the wall drift is determined as follows:

$\Delta_{wall} = 9.32\text{x}10^{-11}(3{,}000)^{2.8} = 0.51$ inches

Note: This analysis, as with most other methods of determining drift, may overlook many factors in the as-built construction that serve to increase or decrease drift. As discussed in Section 6.2, whole building tests seem to confirm that drift is generally over-predicted.

Conclusion

In this example, the determination of the design shear capacity of a segmented shear wall was presented for seismic design and wind design applications. Issues related to connection design for base shear transfer and overturning forces (chord tension and compression) were also discussed and calculations were made to estimate these forces using a conventional design approach. In particular, issues related to capacity-based design and "balanced design" of connections were discussed. Finally, a method to determine the load-drift behavior of a segmented shear wall line was presented. The final design may vary based on designer decisions and judgments (as well as local code requirements) related to the considerations and calculations as given in this example.

EXAMPLE 6.2 *Perforated Shear Wall Design*

Given

The perforated shear wall, as shown in the figure below, is essentially the same wall used in Example 6.1. The following dimensions are used:

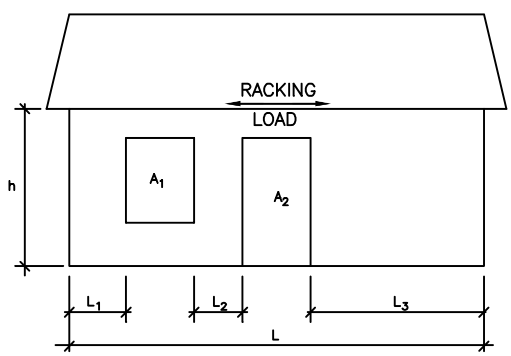
h = 8 ft
L_1 = 3 ft
L_2 = 2 ft
L_3 = 8 ft
L = 19 ft
A_1 = 3.2 ft x 5.2 ft = 16.6 sf (rough window opening area)
A_2 = 3.2 ft x 6.8 ft = 21.8 sf (rough door opening area)

Wall construction:

- Exterior sheathing is 7/16-inch-thick OSB with 8d pneumatic nails (0.113 inch diameter by 2 3/8 inches long) spaced 6 inches on center on panel edges and 12 inches on center in panel field
- Interior sheathing is 1/2-inch-thick gypsum wall board with #6 screws at 12 inches on center
- Framing lumber is Spruce-Pine-Fir, Stud grade (specific gravity, G = 0.42); studs are spaced at 16 inches on center.

Loading condition (assumed for illustration):

Wind shear load on wall line = 3,000 lb
Seismic shear load on wall line = 1,000 lb



Find

1. Design capacity of the perforated shear wall line for wind and seismic shear resistance.
2. Base shear connection requirements.
3. Chord tension and compression forces.
4. Load-drift behavior of the perforated shear wall line and estimated drift at design load conditions.

Solution

1. Determine the factored and adjusted (design) shear capacity for the perforated shear wall line.

$F'_s = F_s\ C_{sp}\ C_{ns}\ [1/SF]$ (Eq. 6.5-1a)

$C_{sp} = [1-(0.5-0.42)] = 0.92$ (Section 6.5.2.3)
$C_{ns} = 0.75$ (Table 6.7)
$SF = 2.0$ (wind design) or 2.5 (seismic design) (Table 6.5)

$F_s = F_{s,ext} + F_{s,int}$ (Section 6.5.2.1)

$F_{s,ext} = 905$ plf (Table 6.1)
$F_{s,int} = 80$ plf (Table 6.3)

For wind design

$F_{s,wind} = 905\text{ plf} + 80\text{ plf} = 985\text{ plf}$
$F'_{s,wind} = (985\text{ plf})(0.92)(0.75)(1/2.0) = 340\text{ plf}$

For seismic design

$F_{s,seismic} = 905\text{ plf} + 0\text{ plf} = 905\text{ plf}$
$F'_{s,seismic} = (905\text{ plf})(0.92)(0.75)(1/2.5) = 250\text{ plf}$

The design capacity of the perforated shear wall is now determined as follows:

$F_{psw} = F'_s\ C_{op}\ C_{dl}\ L$ (Eq. 6.5-1b)

where,

$C_{op} = r/(3-2r)$
$r = 1/(1+\alpha/\beta)$
$\alpha = \Sigma A_o/(h \times L) = (A_1 + A_2)/(h \times L)$
$= (16.6\text{ sf} + 21.8\text{ sf})/(8\text{ ft})(19\text{ ft}) = 0.25$
$\beta = \Sigma L_i/L = (L_1 + L_2 + L_3)/L$
$= (3\text{ ft} + 2\text{ ft} + 8\text{ ft})/(19\text{ ft}) = 0.68$
$r = 1/(1+0.25/0.68) = 0.73$
$C_{op} = 0.73/(3-2(0.73)) = 0.47$

$C_{dl} = 1 + 0.15(w_D/300) \leq 1.15$

Assume for the sake of this example that the roof dead load supported at the top of the wall is 225 plf and that the design wind uplift force on the top of the wall is 0.6(225 plf) – 400 plf = -265 plf (net design uplift). Thus, for wind design in this case, no dead load can be considered on the wall and the C_{dl} factor does not apply for calculation of the perforated shear wall resistance to wind loads. It does apply to seismic design, as follows:

w_D = 0.6*(225 plf) = 135 plf

*The 0.6 factor comes from the load combinations 0.6D + (W or 0.7E) or 0.6D – W_u as given in Chapter 3.

C_{dl} = 1 + 0.15(135/300) = 1.07

For wind design,

$F_{psw,wind}$ = (340 plf)(0.47)(1.0)(19 ft) = 3,036 lb

For seismic design,

$F_{psw,seismic}$ = (250 plf)(0.47)(1.07)(19 ft) = 2,389 lb

Note: In Example 6.1 using the segmented shear wall approach, the design shear capacity of the wall line was estimated as 4,237 lb (wind) and 3,010 lb (seismic) when all of the segments were restrained against overturning by use of hold-down devices. However, given that the design shear load on the wall is 3,000 lb (wind) and 1,000 lb (seismic), the perforated shear wall design capacity as determined above is adequate, although somewhat less than that of the segmented shear wall. Therefore, hold-downs are only required at the wall ends (see Step 3).

2. Determine the base shear connection requirement for the perforated shear wall.

If the wall had a continuous bottom plate that serves as a distributor of the shear forces resisted by various portions of the wall, the base shear connection could be based on the perforated shear wall's design capacity as determined in Step 1 as follows:

For wind design,

Uniform base shear = (3,036 lb)/19 ft = 160 plf

For seismic design,

Uniform base shear = (2,389 lb)/19 ft = 126 plf

However, the wall bottom plate is not continuous in this example and, therefore, the base shears experienced by the portions of the wall to the left and right of the door opening are different as was the case in the segmented shear wall design approach of Example 6.1. As a conservative solution, the base shear connection could be designed to resist the design unit shear capacity of the wall construction, $F'_{s,wind}$ = 340 plf or $F'_{s,seismic}$ = 250 plf. Newer codes that recognize the perforated shear method may require this more conservative approach to be used when the bottom plate is not continuous such that it serves as a distributor (i.e., similar in function to a shear wall collector except shear transfer is out of the wall instead of into the wall). Of course, the bottom plate must be continuous and any splices must be adequately detailed in a fashion similar to collectors (see Example 6.3).

As an alternative, the portion of the wall to the left of the door opening can be treated as a separate perforated shear wall for the left-to-right loading condition. In doing so, the design shear capacity of the left portion of the wall may be determined to be 1,224 lb and the base shear connection required is (1,224 lb)/8ft = 153 plf, much less than the 340 lb required in the wind load condition. The right side of the wall is solid sheathed and, for the right-to-left loading condition, the base shear is equivalent to the design shear capacity of the wall or 340 plf. These calculations can also be performed using the seismic design values for the perforated shear wall. This approach is based on the behavior of a perforated shear wall where the leading edge and the immediately adjacent shear wall segments are fully restrained as in the segmented shear wall approach for one direction of loading. Thus, these segments will realize their full unit shear capacity for one direction of loading. Any interior segments will contribute, but at a reduced amount do to the reduced restraint condition. This behavior is represented in the adjustment provided by the C_{op} factor which is the basis of the perforated shear wall method. Unfortunately, the exact distribution of the uplift forces and shear forces within the wall are not known. It is for this reason that they are assigned conservative values for design purposes. Also, to accommodate potential uplift forces on the bottom plate in the regions of interior perforated shear wall segments, <u>the base shear connections are required to resist an uplift load equivalent to the design unit shear capacity of the wall construction</u>. In the case of this example, the base shear connection would need to resist a shear load of 340 plf (for the wind design condition) and an uplift force of 340 plf (even if under a zero wind uplift load).

Testing has shown that for walls constructed similar to the one illustrated in this example, a bottom plate connection of 2 16d pneumatic nails (0.131 inch diameter by 3 inches long) at 16 inches on center or 5/8-inch-diameter anchor bolts at 6 feet on center provides suitable shear and uplift resistance – at least equivalent to the capacity of the shear wall construction under conditions of no dead load or wind uplift (NAHBRC, 1999). For other conditions, this connection must be designed following the procedures given in Chapter 7 using the conservative assumptions as stated above.

As an alternative base connection that eliminates the need for hold-down brackets at the ends of the perforated shear wall, straps can be fastened to the individual studs to resist the required uplift force of 340 plf as applicable to this example. If the studs are spaced 16 inches on center, the design capacity of the strap must be (340 plf)(1.33 ft/stud) = 452 lb per stud. If an uplift load due to wind uplift on the roof must also be transferred through these straps, the strap design capacity must be increased accordingly. In this example, the net wind uplift at the top of the wall was assumed to be 265 plf. At the base of the wall, the uplift is 265 plf – 0.6(8 ft)(8 psf) = 227 plf. Thus, the total design uplift restraint must provide 340 plf + 227 plf = 567 plf. On a per stud basis (16 inch on center framing), the design load is 1.33 ft/stud x 567 plf = 754 lb/stud. This value must be increased for studs adjacent to wall openings where the wind uplift force in increased. This can be achieved by using multiple straps or by specifying a larger strap in these locations. Of course, the above combination of uplift loads assumes that the design wind uplift load on the roof occurs simultaneously with the design shear load on the wall. However, this condition is not usually representative of actual conditions depending on wind orientation, building configuration, and the shear wall location relative to the uplift load paths.

3. Determine the chord tension and compression forces

The chord tension and compression forces are determined following the same method as used in Example 6.1 for the segmented shear wall design method, but only for the first wall segment in the perforated shear wall line (i.e. the restrained segment). Therefore, the tension forces at the end of the wall are identical to those calculated in Example 6.1 as shown below:

Left end of the wall (Segment 1 in Example 6.1):

T = 2,947 lb (wind design)
T = 2,093 lb (seismic design)

Right end of the wall (Segment 3 in Example 6.1):

T = 3,004 lb (wind design)
T = 2,133 lb (seismic design)

Note: One tension bracket (hold-down) is required at each the end of the perforated shear wall line and not on the interior segments. Also, refer to the notes in Example 6.1 regarding “balanced design” of overturning connections and base shear connections, particularly when designing for seismic loads.

4. Determine the load-drift behavior of the perforated shear wall line.

The basic equation for load-drift estimation of a perforated shear wall line is as follows (Section 6.5.2.6):

$$\Delta = 1.8\left(\frac{0.5}{G}\right)\left(\frac{1}{\sqrt{r}}\right)\left(\frac{V_d}{F_{PSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \qquad \text{(Eq. 6.5-8)}$$

h = 8 ft
G = 0.42(specific gravity for Spruce-Pine-Fir)
r = 0.73 (sheathing area ratio determined in Step 1)

$F_{psw,ult,wind}$ = $(F_{psw,wind})(SF)$ = (3,036 lb)(2.0) = 6,072 lb
$F_{psw,ult,seismic}$ = $(F_{psw,seismic})(SF)$ = (2,389 lb)(2.5) = 5,973 lb

Substituting in the above equation,

$\Delta_{wind} = 6.4x10^{-11}(V_{d,wind})^{2.8}$
$\Delta_{seismic} = 6.7x10^{-11}(V_{d,seismic})^{2.8}$

For the design wind load of 3,000 lb and the design seismic load of 1,000 lb (assumed for the purpose of this example), the drift is estimated as follows:

$\Delta_{wind} = 6.4x10^{-11}(3{,}000)^{2.8}$ = 0.35 inches
$\Delta_{seismic} = 6.7x10^{-11}(1{,}000)^{2.8}$ = 0.02 inches

Note: The reader is reminded of the uncertainties in determining drift as discussed in Example 6.1 and also in Chapter 6. For seismic design, some codes may require the design seismic drift to be amplified (multiplied by) a factor of 4 to account for the potential actual forces that may be experienced relative to the design forces that are determined using an R factor; refer to Chapter 3 for additional discussion. Thus, the amplified drift may be determined as 4 x 0.02 inches = 0.08 inches. However, if the seismic shear load is magnified (i.e., 4 x 1,000 lb = 4,000 lb) to account for a possible actual seismic load (not modified for the seismic response of the shear wall system), the seismic drift calculated in the above equation becomes 0.8 inches which is an order of magnitude greater. The load adjustment is equivalent to the use of an R of 1.5 instead of 6 in Equation 3.8-1 of Chapter 3. However, this latter approach of magnifying the load is not currently required in the existing building codes for drift determination. As mentioned, drift is not usually considered in residential design. Finally, the above equations may be used to determine a load-drift curve for a perforated shear wall for values of V_d ranging from 0 to $F_{psw,ult}$. While the curve represents the non-linear behavior of a perforated shear wall, it should only be considered as a representation, and not an exact solution.

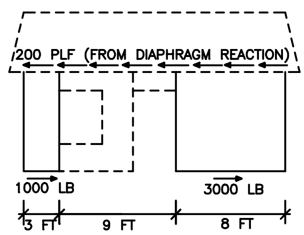
Conclusion

In this example, the determination of the design shear capacity of a perforated shear wall was presented for seismic design and wind design applications. Issues related to connection design for base shear transfer and overturning forces (chord tension) were also discussed and calculations (or conservative assumptions) were made to estimate these forces. In particular, issues related to capacity-based design and "balanced design" of connections were discussed. Finally, a method to determine the load-drift behavior of a perforated shear wall line was presented. The final design may vary based on designer decisions and judgments (as well as local code requirements) related to the considerations and calculations as given in this example.

EXAMPLE 6.3 *Shear Wall Collector Design*

Given

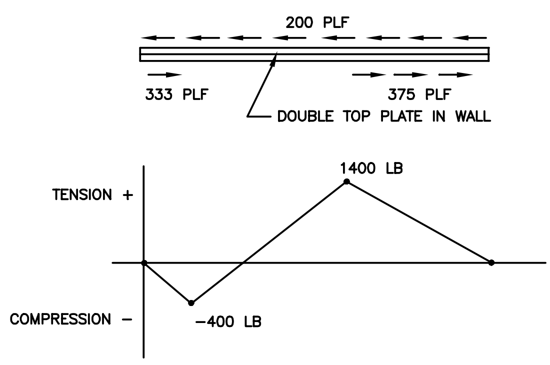
The example shear wall, assumed loading conditions, and dimensions are shown in the figure below.



Find The maximum collector tension force

Solution

1. The collector force diagram is shown below based on the shear wall and loading conditions in the figure above.



The first point at the interior end of the left shear wall segment is determined as follows:

200 plf (3 ft) – 333 plf (3 ft) = - 400 lb (compression force)

The second point at the interior end of the right shear wall segment is determined as follows:

- 400 lb + 200 plf (9 ft) = 1,400 lb (tension force)

The collector load at the right-most end of the wall returns to zero as follows:

1,400 lb – 375 plf (8 ft) + 200 plf (8 ft) = 0 lb

Conclusion

The maximum theoretical collector tension force is 1,400 lb at the interior edge of the 8-foot shear wall segment. The analysis does not consider the contribution of the "unrestrained" wall portions that are not designated shear wall segments and that would serve to reduce the amount of tension (or compression) force developed in the collector. In addition, the load path assumed in the collector does not consider the system of connections and components that may share load with the collector (i.e., wall sheathing and connections, floor or roof construction above and their connections, etc.). Therefore, the collector load determined by assuming the top plate acts as an independent element can be considered very conservative depending on the wall-floor/roof construction conditions. Regardless, it is typical practice to design the collector (and any splices in the collector) to resist a tension force as calculated in this example. The maximum compressive force in the example collector is determined by reversing the loading direction and is equal in magnitude to the maximum tension force. Compressive forces are rarely a concern when at least a double top plate is used as a collector, particularly when the collector is braced against lateral buckling by attachment to other construction (as would be generally necessary to deliver the load to the collector from somewhere else in the building).

EXAMPLE 6.4 *Horizontal (Floor) Diaphragm Design*

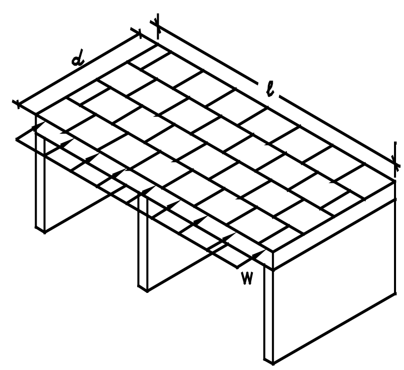
Given

The example floor diaphragm and its loading and support conditions are shown in the figure below. The relevant dimensions and loads are as follows:

d	= 24 ft	
l	= 48 ft	
w	= 200 plf	(from wind or seismic lateral load)*

*Related to the diaphragm's tributary load area; see Chapter 3 and discussions in Chapter 6.

The shear walls are equally spaced and it is assumed that the diaphragm is flexible (i.e. experiences beam action) and that the shear wall supports are rigid. This assumption is not correct because the diaphragm may act as a "deep beam" and distribute loads to the shear wall by "arching" action rather than bending action. Also, the shear walls cannot be considered to be perfectly rigid or to exhibit equivalent stiffness except when designed exactly the same with the same interconnection stiffness and base support stiffness. Regardless, the assumptions made in this example are representative of typical practice.



Find

1. The maximum design unit shear force in the diaphragm (assuming simple beam action) and the required diaphragm construction.
2. The maximum design moment in the diaphragm (assuming simple beam action) and the associated chord forces.

Solution

1.

The maximum shear force in the diaphragm occurs at the center shear wall support. Using the beam equations in Appendix A for a 2-span beam, the maximum shear force is determined as follows:

$$V_{max} = \frac{5}{8}w\left(\frac{l}{2}\right) = \frac{5}{8}(200\ \text{plf})\left(\frac{48\ \text{ft}}{2}\right) = 3{,}000\ \text{lb}$$

The maximum design unit shear in the diaphragm is determined as follows:

$$v_{max} = \frac{V_{max}}{d} = \frac{3{,}000\ \text{lb}}{24\ \text{ft}} = 125\ \text{plf}$$

From Table 6.8, the lightest unblocked diaphragm provides adequate resistance. Unblocked means that the panel edges perpendicular to the framing (i.e., joists or rafters) are not attached to blocking. The perimeter, however, is attached to a continuous member to resist chord forces. For typical residential floor construction a 3/4-inch-thick subfloor may be used which would provide at least 240 plf of design shear capacity. In typical roof construction, a minimum 7/16-inch-thick sheathing is used which would provide about 230 plf of design shear capacity. However, residential roof construction does not usually provide the edge conditions (i.e., continuous band joist of 2x lumber) associated with the diaphragm values in Table 6.8. Regardless, roof diaphragm performance has rarely (if ever) been a problem in light-frame residential construction and these values are often used to approximate roof diaphragm design values.

Note: The shear forces at other regions of the diaphragm and at the locations of the end shear wall supports can be determined in a similar manner using the beam equations in Appendix A. These shear forces are equivalent to the connection forces that must transfer shear between the diaphragm and the shear walls at the ends of the diaphragm. However, for the center shear wall, the reaction (connection) force is twice the unit shear force in the diaphragm at that location (see beam equations in Appendix A). Therefore, the connection between the center shear wall and the diaphragm in this example must resist a design shear load of 2 x 125 plf = 250 plf. However, this load is very dependent on the assumption of a “flexible” diaphragm and “rigid” shear walls.

2. The maximum moment in the diaphragm also occurs at the center shear wall support. Using the beam equations in Appendix A, it is determined as follows:

$$M_{max} = \frac{1}{8} w \left(\frac{l}{2}\right)^2 = \frac{1}{8}(200\text{plf})\left(\frac{48\text{ft}}{2}\right)^2 = 14{,}400\text{ft} - \text{lb}$$

The maximum chord tension and compression forces are at the same location and are determined as follows based on the principle of a force couple that is equivalent to the moment:

$$T = C = \frac{M_{max}}{d} = \frac{14{,}400\text{ft} - \text{lb}}{24\text{ft}} = 600\text{lb}$$

Therefore, the chord members (i.e., band joist and associated wall or foundation framing that is attached to the chord) and splices must be able to resist 600 lb of tension or compression force. Generally, these forces are adequately resisted by the framing systems bounding the diaphragm. However, the adequacy of the chords should be verified by the designer based on experience and analysis as above.

Conclusion

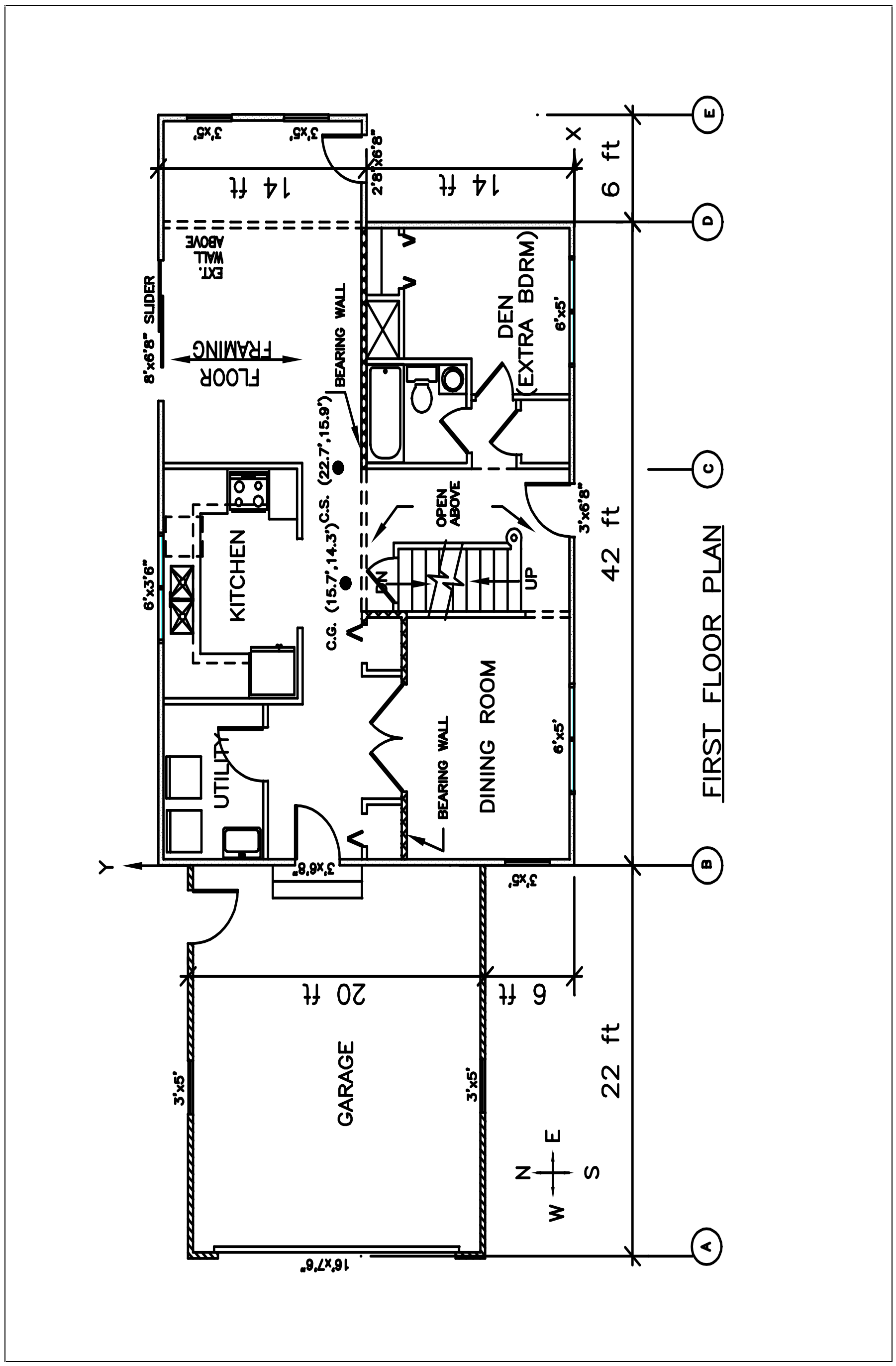
In this example, the basic procedure and principles for horizontal diaphragm design were presented. Assumptions required to conduct a diaphragm analysis based on conventional beam theory were also discussed.

EXAMPLE 6.5 *Horizontal Shear Load Distribution Methods*

Given

General

In this example, the first floor plan of a typical two-story house with an attached garage (see Figure below) is used to demonstrate the three methods of distributing shear loads discussed in Chapter 6, Section 6.4.2. The first story height is 8 ft (i.e., 8 ft ceiling height). Only the load in the North-South (N-S) direction is considered in the example. In a complete design, the load in the East-West (E-W) direction would also need to be considered.



Lateral Load Conditions

The following design N-S lateral loads are determined for the story under consideration using the methods described in Chapter 3 for wind and seismic loads. A fairly high wind load and seismic load condition is assumed for the purpose of the example.

Design N-S Wind Lateral Load (120 mph gust, exposure B)

House:	17,411	lb total story shear
Garage:	3,928	lb total story shear
Total:	21,339	lb

Design N-S Seismic Lateral Load (mapped S_s = 1.5g)

House:	7,493	lb total story shear (tributary weight is 37,464 lb)
Garage:	1,490	lb total story shear (tributary weight is 7,452 lb)
Total:	8,983	lb

Designation of Shear Walls in N-S Direction

Initially, there are four N-S lines designated in the first story for shear wall construction. The wall lines are A, B, D, and E. If needed, an interior wall line may also be designated and designed as a shear wall (see wall line C in the figure above).

The available length of full-height wall segments in each N-S shear wall line is estimated as follows from the floor plan:

Wall Line A:	2 ft + 2 ft	= 4 ft	(garage return walls)
Wall Line B:	~~1.33 ft~~* + 11 ft + 9 ft	= 20 ft	(garage/house shared wall)
Wall Line D:	14 ft	= 14 ft	(den exterior wall)
Wall Line E:	2 ft + 3 ft + 2 ft	= 7 ft	(living room exterior wall)
Total:		= 45 ft	

*The narrow 1.33 ft segment is not included in the analysis due to the segment's aspect ratio of 8 ft/1.33 ft = 6, which is greater than the maximum allowable of 4. Some current building codes may restrict the segment aspect ratio to a maximum of 2 or 3.5 depending on the code and the edition in local use. In such a case, many of the useable shear wall segments would be eliminated (i.e., all of the 2 ft segments). Thus, the garage opening wall would require larger segments, a portal frame (see Section 6.5.2.7), or transfer of the garage shear load to the house by torsion (i.e., treat the garage as a cantilever projecting from the house under a uniform lateral load).

Find

1. Using the "total shear method" of horizontal shear load distribution, determine the total length of shear wall required and the required shear wall construction in the N-S direction.
2. Using the "tributary area method" of horizontal shear load distribution, determine the shear resistance and wall construction required in each N-S shear wall line.
3. Using the "relative stiffness method" of horizontal shear load distribution, determine the shear loads on the N-S shear wall lines.

Solution

1. Using the total shear approach, determine the unit shear capacity required based on the given amount of available shear wall segments in each N-S wall line and the total N-S shear load.

In this part of the example, it is assumed that the wall lines will be designed as segmented shear wall lines. From the given information, the total length of N-S shear wall available is 45 ft. It is typical practice in this method to not include segments with aspect ratios greater than 2 since stiffness effects on the narrow segments are not explicitly considered. This would eliminate the 2 ft segments and the total available length of shear wall would be 45 ft – 8 ft = 37 ft in the N-S direction.

The required design unit shear capacity of the shear wall construction and ultimate capacity is determined as follows for the N-S lateral design loads:

<u>Wind N-S</u>

$F'_{s,wind}$ = (21,339 lb)/37 ft = 576 plf
$F_{s,\ wind} = (F'_{s,wind})(SF)$ = (576 plf)(2.0) = 1,152 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the shear walls must meet or exceed 1,152 plf. Assuming that standard 1/2-thick GWB finish is used on the interior wall surfaces (80 plf minimum from Table 6.3), the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,wind} = F_{s,ext} + F_{s,int}$
$F_{s,ext}$ = 1,152 plf – 80 plf = 1,072 plf

From Table 6.1, any of the wall constructions that use a 4 inch nail spacing at the panel perimeter exceed this requirement. By specifying and 3/8-thick Structural I wood structural panel with 8d common nails spaced at 4 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors. For example, assume the following framing lumber and nails are used in the shear wall construction:

lumber species:	Spruce-Pine-Fir (G=0.42)	C_{sp} = 0.92
nail type:	8d pneumatic, 0.113-inch-diameter	C_{ns} = 0.75

Thus, values in Table 6.1 would need to be multiplied by (0.92)(0.75) = 0.69. This adjustment requires a 15/32-inch-thick sheathing with the 8d nails (i.e., 1,539 plf x 0.69 = 1,062 plf which is close enough to the required 1,072 plf for practical design purposes). Alternatively, a 7/16-inch thick wood structural panel sheathing could be used in accordance with footnote 5 of Table 6.1; however, the horizontal joint between panels would need to be blocked. In extreme lateral load conditions, it may be necessary (and more efficient) to consider a "double sheathed" wall construction (i.e., structural wood panels on both sides of the wall framing) or to consider the addition of an interior shear wall line (i.e., design the interior walls along wall line C as shear walls).

Seismic N-S

$F'_{s,seismic}$ = (8,983 lb)/37 ft = 243 plf
$F_{s,\ seismic}$ = ($F'_{s,seismic}$)(SF) = (243 plf)(2.5) = 608 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the wall line must meet or exceed 608 plf. Since seismic codes do not permit the consideration of a 1/2-thick GWB interior finish, the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,seismic} = F_{s,ext}$ = 608 plf

From Table 6.1, any of the wood structural panel wall constructions that use a 6 inch nail spacing at the panel perimeter exceed this requirement. By specifying 3/8-inch-thick Structural I wood structural panels with 8d common nails spaced at 6 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors as demonstrated above for the N-S wind load case.

The base shear connections may be designed in this method by considering the total length of continuous bottom plate in the N-S shear wall lines. As estimated from the plan, this length is approximately 56 feet. Thus, the base connection design shear load (parallel to the grain of the bottom plate) is determined as follows:

Base wind design shear load = (21,339 lb)/(56 ft) = 381 plf
Base seismic design shear load = (8,983 lb)/(56 ft) = 160 plf

The base shear connections may be designed and specified following the methods discussed in Chapter 7 – Connections. A typical 5/8-inch-diameter anchor bolt spaced at 6 feet on center or standard bottom plate nailing may be able to resist as much as 800 plf (ultimate shear capacity) which would provided a "balanced" design capacity of 400 plf or 320 plf for wind and seismic design with safety factors of 2.0 and 2.5, respectively. Thus, a conventional wall bottom plate connection may be adequate for the above condition; refer to Chapter 7 for connection design information and the discussion in Section 7.3.6 for more details on tested bottom plate connections.

If the roof uplift load is not completely offset by 0.6 times the dead load at the base of the first story wall, then strapping to transfer the net uplift from the base of the wall to the foundation or construction below must be provided.

The hold-down connections for the each shear wall segment in the designated shear wall lines are designed in the manner shown in Example 6.1. Any overturning forces originating from shear walls on the second story must also be included as described in Section 6.4.2.4.

Notes:

1. The contribution of the interior walls to the lateral resistance is neglected in the above analysis for wind and seismic loading. As discussed in Chapter 6, these walls can contribute significantly to the lateral resistance of a home and serve to reduce the designated shear wall loads and connection loads through alternate, "non-designed" load paths. In this example, there is approximately 40 ft of interior partition walls in the N-S direction that each have a minimum length of about 8 ft or more (small segments not included). Assuming a design unit shear value of 80 plf / 2 = 40 plf (safety factor of 2), the design lateral resistance may be at least 40 ft x 40 plf = 1,600 lb. While this is not a large amount, it should factor into the design consideration, particularly when a lateral design solution is considered to be marginal based on an analysis that does not consider interior partition walls.
2. Given the lower wind shear load in the E-W direction, the identical seismic story shear load in the E-W direction, and the greater available length of shear wall in the E-W direction, an adequate amount of lateral resistance should be no problem for shear walls in the E-W direction. It is probable that some of the available E-W shear wall segments may not even be required to be designed and detailed as shear wall segments. Also, with hold-down brackets at the ends of the N-S walls that are detailed to anchor a common corner stud (to which the corner sheathing panels on each wall are fastened with the required panel edge fastening), the E-W walls are essentially perforated shear wall lines and may be treated as such in evaluating the design shear capacity of the E-W wall lines.
3. The distribution of the house shear wall elements appears to be reasonably "even" in this example. However, the garage opening wall could be considered a problem if sufficient connection of the garage to the house is not provided to prevent the garage from rotating separately from the house under the N-S wind or seismic load. Thus, the garage walls and garage roof diaphragm should be adequately attached to the house so that the garage and house act as a structural unit. This process will be detailed in the next part of this example.

2. Determine the design shear load on each wall line based on the tributary area method.

Following the tributary area method of horizontal force distribution, the loads on the garage and the house are treated separately. The garage lateral load is assumed to act through the center of the garage and the house load is assumed to act through the center of the house. The extension of the living room on the right side of the plan is only one story and is considered negligible in its impact to the location of the real force center; although, this may be considered differently by the designer. Therefore, the lateral force (load) center on the garage is considered to act in the N-S direction at a location one-half the distance between wall lines A and B (see the given floor plan diagram). Similarly, the N-S force center on the house may be considered to act half-way between wall lines B and D (or perhaps a foot or less farther to the right to compensate for the living room "bump-out"). Now, the N-S lateral design loads are assigned to wall lines A, B, and D/E as follows:

<u>Wall Line A</u>

Wind design shear load = 1/2 garage shear load = 0.5(3,928 lb) = 1,964 lb
Seismic design shear load = 0.5(1,490 lb) = 745 lb

<u>Wall Line B</u>

Wind design shear load = 1/2 garage shear load + 1/2 house shear load
= 1,964 lb + 0.5(17,411 lb) = 10,670 lb
Seismic design shear load = 745 lb + 0.5(7,493 lb) = 4,492 lb

Wall Line D/E

Wind design shear load = 1/2 house shear load = 0.5(17,411 lb) = 8,706 lb
Seismic design shear load = 0.5(7,493 lb) = 3,747 lb

Based on the design shear loads above, each of the wall lines may be designed in a fashion similar to that used in Step 1 (total shear method) by selecting the appropriate wall construction to meet the loading demand. For example, the design of wall line B would proceed as shown below (using the perforated shear wall method in this case) for the required wind shear load.

The following equations are used to determine the required ultimate shear capacity, F_s, of the wall construction (interior and exterior sheathing type and fastening):

$F'_s = [(F_{s,ext})(C_{sp})(C_{ns}) + F_{s,int}]x[1/SF]$ (based on Eq. 6.5-1a)
$F_{psw} = F'_s\, C_{op}\, C_{dl}\, [L]$ (Eq. 6.5-1b)

Substituting the first equation above into the second,

$F_{psw} = [(F_{s,ext})(C_{sp})(C_{ns}) + F_{s,int}]\, [1/SF]\, C_{op}\, C_{dl}\, [L]$

To satisfy the design wind shear load requirement for Wall Line B,

$F_{psw} \geq 10{,}670$ lb

Assume that the wall construction is the same as used in Example 6.2. The following parameters are determined for Wall Line B:

$C_{sp} = 0.92$	(Spruce-Pine-Fir)
$C_{ns} = 0.75$	(8d pneumatic nail, 0.113-inch-diameter)
$C_{dl} = 1.0$	(zero dead load due to wind uplift)
SF = 2.0	(wind design safety factor)
$C_{op} = 0.71$	(without the corner window and narrow segment)*
L = 28 ft – 1.33 ft – 3 ft = 23.67 ft	(length of perforated shear wall line)*
$F_{s,int} = 80$ plf	(Table 6.3, minimum ultimate unit shear capacity)

*The perforated shear wall line begins at the interior edge of the 3’ x 5’ window opening because the wall segment adjacent to the corner exceeds the maximum aspect ratio requirement of 4. Therefore, the perforated shear wall is “embedded” in the wall line.

Substituting the values above into the equation for F_{psw} , the following value is obtained for $F_{s,ext}$:

$10{,}670 \text{ lb} = [(F_{s,ext})(0.92)(0.75) + 80 \text{ plf}]\, [1/2.0]\, (0.71)\, (1.0)\, [23.67 \text{ ft}]$

$F_{s,ext} = 1{,}724$ plf

By inspection in Table 6.1, the above value is achieved for a shear wall constructed with 15/32-inch-thick Structural 1 wood structural panel sheathing with nails spaced at 3 inches on the panel edges. The value is 1,722 plf which is close enough for practical purposes (particularly given that contribution of interior walls is neglected in the above analysis). Also, a thinner sheathing may be used in accordance with Footnote 5 of Table 6.1. As another alternative, wall line B could be designed as a segmented shear wall. There are two large shear wall segments that may be used. In total they are 20 ft long. Thus, the required ultimate shear capacity for wall line B using the segmented shear wall method is determined as follows:

$F'_s = (F_{s,ext}\, C_{sp}\, C_{ns} + F_{s,int})\, C_{ar}\, [1/SF]$ (based on Eq. 6.5-2a)
$F_{ssw} = F'_s \times L$ (Eq. 6.5-2b)
$F_{ssw} \geq 10{,}670$ lb (wind load requirement on wall line B)

Substituting the first equation into the second

$F_{ssw} = (F_{s,ext}\, C_{sp}\, C_{ns} + F_{s,int})\, C_{ar}\, [1/SF] \times L$

The following parameter values are used:

$C_{sp} = 0.92$ (same as before)
$C_{ns} = 0.75$ (same as before)
$C_{ar} = 1.0$ (both segments have aspect ratios less than 2)*
SF = 2.0 (for wind design)
L = 20 ft (total length of the two shear wall segments)*
$F_{s,int} = 80$ plf (minimum ultimate unit shear capacity)

*If the wall segments each had different values for C_{ar} because of varying adjustments for aspect ratio, then the segments must be treated independently in the equation above and the total length could not be summed as above to determine a total L.

Now, solving the above equations for $F_{s,ext}$ the following is obtained:

10,670 lb = [($F_{s,ext}$)(0.92)(0.75) + 80 plf](1.0)[1/2.0](20 ft)

$F_{s,ext} = 1{,}430$ plf

By inspection of Table 6.1 using the above value of $F_{s,ext}$, a 4 inch nail spacing may be used to meet the required shear loading in lieu of the 3 inch nail spacing used if the wall were designed as a perforated shear wall. However, two additional hold down brackets would be required in Wall Line B to restrain the two wall segments as required by the segmented shear wall design method.

Wall Line A poses a special design problem since there are only two narrow shear wall segments to resist the wind design lateral load (1,964 lb). Considering the approach above for the segmented shear wall design of Wall Line B and realizing that $C_{ar} = 0.71$ (aspect ratio of 4), the following value for $F_{s,ext}$ is obtained for Wall Line A:

$F_{ssw} = (F_{s,ext}\, C_{sp}\, C_{ns} + F_{s,int})\, C_{ar}\, [1/SF] \times L$

1,964 lb = [($F_{s,ext}$)(0.92)(0.75) + 0*](0.71)[1/2.0](4 ft)

*The garage exterior walls are assumed not to have interior finish. The shared wall between the garage and the house, however, is required to have a fire rated wall which is usually satisfied by the use of 5/8-thick gypsum wall board. This fire resistant finish is placed over the wood structural sheathing in this case and the impact on wall thickness (i.e. door jamb width) should be considered by the architect and builder.

Solving for $F_{s,ext}$,

$F_{s,ext} = 2{,}004$ plf

By inspecting Table 6.1, this would require 15/32-inch-thick wood structural panel with nails spaced at 2 inches on center and would require 3x framing lumber (refer to footnote 3 of Table 6.1). However, the value of C_{ns} (=0.75) from Table 6.7 was based on a 0.113-inch diameter nail for which the table does not give a conversion relative to the 10d common nail required in Table 6.1. Therefore, a larger nail should be used at the garage opening. Specifying an 8d common nail or similar pneumatic nail with a diameter of 0.131 inches (see Table 6.7), a C_{ns} value of 1.0 is used and $F_{s,ext}$ may be recalculated as above to obtain the following:

$$F_{ssw} = (F_{s,ext}\ C_{sp}\ C_{ns} + F_{s,int})\ C_{ar}\ [1/SF] \times L$$

$$1{,}964 \text{ lb} = [(F_{s,ext})(0.92)(1.0) + 0](0.71)[1/2.0](4 \text{ ft})$$

$$F_{s,ext} = 1{,}503 \text{ plf}$$

Inspecting Table 6.1 again, it is now found that 15/32-inch-thick wood structural panel sheathing with 8d common nails spaced at 4 inches on center provides an ultimate rated unit shear capacity of 1,539 plf > 1,503 plf. This design does not require the use of 3x framing lumber which allows the same lumber to be used for all wall construction. The only added detail is the difference in nail type and spacing for the garage return walls. From the standpoint of simplicity, the easiest solution would be to increase the width of the garage shear wall segments; however, design simplicity is not always the governing factor. Also, a portal frame system may be designed based on the information and references provided in Section 6.5.2.7.

Finally, the garage should be adequately tied to the building to ensure that the garage section and the house section act as a structural unit. This may be achieved by fastening the end rafter or truss top chord in the roof to the house framing using fasteners with sufficient withdrawal capacity (i.e. ring shank nails or lag screws). The same should be done for the end studs that are adjacent to the house framing. Ideally, the garage roof diaphragm may be tied into the house second floor diaphragm by use of metal straps and blocking extending into the floor diaphragm and garage roof diaphragm a sufficient distance in each direction (i.e., 4 feet). With sufficient connection to the house end wall and floor diaphragm, the garage opening issue may be avoided completely. The connection load to the house discussed above can then be determined by treating the garage roof diaphragm as a cantilevered horizontal beam on the side of the home with a fixed end moment at the connection to the house. The fixed end moment (assuming the garage opening provides no lateral shear resistance) is determined based on the beam equation for a cantilever beam (see Appendix A). For the wind load on the garage, the fixed end moment due to lateral load is (3,928 lb)(11 ft) = 43,208 ft-lb. This moment may be resisted by a strap at either side of the garage roof with about a 2,500 lb design tension capacity (i.e. 43,208 ft-lb/18 ft = 2,400 lb). Preferably, the strap would be anchored to the garage roof diaphragm and house floor diaphragm as described above. Alternatively, this moment could be resisted by numerous lag screws or similar fasteners attaching the garage framing to the house framing. By this method, the garage end walls would require no special shear wall design. Of course, connections required to resist wind uplift and transverse shear loads on the garage door and return walls would still be required.

For the seismic design lateral loads in this example, the garage opening is not so severely loaded. The design seismic load on the Wall Line A is 745 lb. Using the approach above (and substituting a safety factor of 2.5 for seismic design), the value of $F_{s,ext}$ determined is 905 plf which is much less than the 2,004 plf determined for the design wind shear load condition assumed in this example. By inspecting Table 6.1, 7/16-inch-thick Structural 1 sheathing is sufficient and the pnuematic nails used on the rest of the building's shear walls may be used. However, this requires the two garage return walls to be restrained with two hold-down brackets each as in the segmented shear wall design method. For the seismic load, the garage opening wall (Wall Line A) may be suitably designed as a perforated shear wall and eliminate the need for two of the four hold-downs. A portal frame may also be considered for the garage opening (see Section 6.5.2.7).

Wall Line D/E may be designed in a similar fashion to the options discussed above. In fact, Wall Line E may be eliminated as a designed shear wall line provided that a collector is provided to bring the diaphragm shear load into the single wall segment in wall line D (see the dotted line on the floor plan figure). Of course, Wall line D must be designed to carry the full design shear load assigned to that end of the building. Collector design was illustrated in Example 6.3. The connections for overturning (i.e., hold-downs) and base shear transfer must be designed as illustrated in Examples 6.1 and 6.2. As an additional option, Wall Line C may be designed as an interior shear wall line and the wood structural panel sheathing would be placed underneath the interior finish. This last option would relieve some of the load on the house end walls and possibly simplify the overall shear wall construction details used in the house.

3. Determine the shear loads on the N-S shear wall lines using the relative stiffness method and an assumed shear wall construction for the given seismic design condition only.

Assume that the shear wall construction will be as follows:

- 7/16-inch OSB Structural I wood structural panel sheathing with 8d common nails (or 0.131-inch diameter 8d pneumatic nails) spaced at 4 inches on center on the panel edges and 12 inches in the panel field.
- Douglas-fir wall framing is used with 2x studs spaced at 16 inches on center.
- Walls are designed as perforated shear wall lines and adequate hold-downs and base shear connections are provided.

It will be further assumed that the house and garage are sufficiently tied together to act as a structural unit. It must be remembered that the relative stiffness design approach is predicated on the assumption that the horizontal diaphragm is rigid in comparison to the supporting shear walls so that the forces are distributed according to the relative stiffness of the shear wall lines. This assumption is exactly opposite to that assumed by use of the tributary area method.

As given for the design example, the following design seismic shear loads apply to the first story of the example building:

Design N-S Seismic Lateral Load (mapped S_s = 1.5g)

House: 7,493 lb total story shear (tributary weight is 37,464 lb)
Garage: 1,490 lb total story shear (tributary weight is 7,452 lb)
Total: 8,983 lb total story shear (total tributary weight is 44,916 lb)

Locate the center of gravity

The first step is to determine the center of gravity of the building at the first story level. The total seismic story shear load will act through this point. For wind design, the process is similar, but the horizontal wind forces on various portions of the building (based on vertical projected areas and wind pressures) are used to determine the force center for the lateral wind loads (i.e., the resultant of the garage and house lateral wind loads).

Establishing the origin of an x-y coordinate system at the bottom corner of Wall Line B of the example first floor plan, the location of the center of gravity is determined by taking weighted moments about each coordinate axis using the center of gravity location for the garage and house portions. Again, the “bump-out” area in living room is considered to have negligible impact on the estimate of the center of gravity since most of the building mass is originating from the second story and roof which does not have the “bump-out” in the plan.

The center of gravity of the garage has the (x,y) coordinates of (-11 ft, 16 ft). The center of gravity of the house has the coordinates (21 ft, 14 ft).

Weighted moments about the y-axis:

$X_{cg,building}$ = [($X_{cg,garage}$)(garage weight) + ($X_{cg,house}$)(house weight)]/(total weight)
= [(-11 ft)(7,452 lb) + (21 ft)(37,464 lb)]/(44,916 lb)
= 15.7 ft

Weighted moments about the x-axis:

$Y_{cg,building}$ = [($Y_{cg,garage}$)(garage weight) + ($Y_{cg,house}$)(house weight)]/(total weight)
= [(16 ft)(7,452 lb) + (14 ft)(37,464 lb)]/(44,916 lb)
= 14.3 ft

Thus, the center of gravity for the first story is located at the (x,y) coordinates of (15.7 ft, 14.3 ft). The approximate location on the floor plan is about 4 inches north of the center bearing wall line and directly in front of the stair well leading down (i.e., about 5 feet to the left of the center of the house).

Locate the center of resistance

The center of resistance is somewhat more complicated to determine and requires an assumption regarding the shear wall stiffness. Two methods of estimating the relative stiffness of segmented shear walls are generally recognized. One method bases the segmented shear wall stiffness on it's length. Thus, longer shear walls have greater stiffness (and capacity). However, this method is less appealing when multiple segments are included in one wall line and particularly when the segments have varying aspect ratios, especially narrow aspect ratios which affect stiffness disproportionately to the length. The second method bases the segmented shear wall stiffness on the shear capacity of the segment, which is more appealing when various shear wall constructions are used with variable unit shear values and when variable aspect ratios are used, particularly when the unit shear strength is corrected for narrow aspect ratios. The method based on strength is also appropriate for use with the perforated shear wall method, since the length of a perforated shear wall has little to do with its stiffness or strength. Rather, the amount of openings in the wall (as well as its construction) govern its stiffness and capacity. Therefore, the method used in this example will use the capacity of the perforated shear wall lines as a measure of relative stiffness. The same technique may be used with a segmented shear wall design method by determining the shear capacity of each shear wall line (comprised of one or more shear wall segments) as shown in Example 6.1.

First, the strength of each shear wall line in the building must be determined. Using the perforated shear wall method and the assumed wall construction given at the beginning of Step 3, the design shear wall line capacities (see below) are determined for each of the exterior shear wall lines in the building. The window and door opening sizes are shown on the plan so that the perforated shear wall calculations can be done as demonstrated in Example 6.2. It is assumed that no interior shear wall lines will be used (except at the shared wall between the garage and the house) and that the contribution of the interior partition walls to the stiffness of the building is negligible. As mentioned, this assumption can overlook a significant factor in the lateral resistance and stiffness of a typical residential building.

PSW 1: F_{psw1} = 7,812 lb (Wall Line D)
PSW 2: F_{psw2} = 3,046 lb (Wall Line E)
PSW 3: F_{psw3} = 14,463 lb (North side wall of house)
PSW 4: F_{psw4} = 9,453 lb (North side of garage)
PSW 5: F_{psw5} = 182 lb (Wall Line A; garage opening)
PSW 6: F_{psw6} = 9,453 lb (South side wall of garage)
PSW 7: F_{psw7} = 9,687 lb (Wall Line B)
PSW 8: F_{psw8} = 11,015 lb (South side wall of house at front)

The center of stiffness on the y-coordinate is now determined as follows using the above PSW design shear capacities for wall lines oriented in the E-W direction:

$$
\begin{aligned}
Y_{cs} &= [(F_{psw3})(Y_{psw3}) + (F_{psw4})(Y_{psw4}) + (F_{psw6})(Y_{psw6}) + (F_{psw8})(Y_{psw8})]/(F_{psw,E\text{-}W}) \\
&= [(14{,}463 \text{ lb})(28 \text{ ft})+(9{,}453 \text{ lb})(26 \text{ ft})+(9{,}453 \text{ lb})(6 \text{ ft})+(11{,}015 \text{ lb})(0 \text{ ft})]/(44{,}384 \text{ lb}) \\
&= 15.9 \text{ ft}
\end{aligned}
$$

The center of stiffness on the x-coordinate is determined similarly considering the wall lines oriented in the N-S direction:

$$
\begin{aligned}
X_{cs} &= [(F_{psw1})(X_{psw1}) + (F_{psw2})(X_{psw2}) + (F_{psw5})(X_{psw5}) + (F_{psw7})(X_{psw7})]/(F_{psw,N\text{-}S}) \\
&= [(7{,}812 \text{ lb})(42 \text{ ft})+(3{,}046 \text{ lb})(48 \text{ ft})+(182 \text{ lb})(-22 \text{ ft})+(9{,}687 \text{ lb})(0 \text{ ft})]/(20{,}727 \text{ lb}) \\
&= 22.7 \text{ ft}
\end{aligned}
$$

Therefore, the coordinates of the center of stiffness are (22.7 ft, 15.9 ft). Thus, the center of stiffness is located to the right of the center of gravity (force center for the seismic load) by 22.7 ft – 15.7 ft = 7 ft. This offset between the center of gravity and the center of resistance will create a torsional response in the N-S seismic load direction under consideration. For E-W seismic load direction, the offset (in the y-coordinate direction) is only 15.9 ft – 14.3 ft = 1.6 ft which is practically negligible from the standpoint of torsional response. It should be remembered that, in both loading directions, the influence of interior partitions on the center of stiffness (and thus the influence on torsional response) is not considered. To conservatively account for this condition and for possible error in locating the actual center of gravity of the building (i.e., accidental torsion), codes usually require that the distance between the center of gravity and the center of stiffness be considered as a minimum of 5 percent of the building dimension perpendicular to the direction of seismic force under consideration. This condition is essentially met in this example since the offset dimension for the N-S load direction is 7 feet which is 10 percent of the E-W plan dimension of the house and attached garage.

<u>Distribute the direct shear forces to N-S walls</u>

The direct shear force is distributed to the N-S walls based on their relative stiffness without regard to the location of the center of stiffness (resistance) and the center of gravity (seismic force center), or the torsional load distribution that occurs when they are offset from each other. The torsional load distribution is superimposed on the direct shear forces on the shear wall lines in the next step of the process.

The direct seismic shear force of 8,983 lb is distributed as shown below based on the relative stiffness of the perforated shear wall lines in the N-S direction. As before, the relative stiffness is based on the design shear capacity of each perforated shear wall line relative to that of the total design capacity of the N-S shear wall lines.

Direct shear on PSW1, PSW2, PSW5, and PSW7 is determined as follows:

(total seismic shear load on story)[(F_{psw1})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(7,812 lb)/(20,727 lb)]
= (8,983 lb)[0.377]
= 3,387 lb

(total seismic shear load on story)[(F_{psw2})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(3,046 lb)/(20,727 lb)]
= (8,983 lb)[0.147]
= 1,321 lb

(total seismic shear load on story)[(F_{psw5})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(182 lb)/(20,727 lb)]
= (8,983 lb)[0.009]
= 81 lb

(total seismic shear load on story)[(F_{psw7})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(9,687 lb)/(20,727 lb)]
= (8,983 lb)[0.467]
= 4,195 lb

Distribute the torsion load

The torsional moment is created by the offset of the center of gravity (seismic force center) from the center of stiffness or resistance (also called the center of rigidity). For the N-S load direction, the torsional moment is equal to the total seismic shear load on the story multiplied by the x-coordinate offset of the center of gravity and the center of stiffness (i.e., 8,983 lb x 7 ft = 62,881 ft-lb). The sharing of this torsional moment on all of the shear wall lines is based on the torsional moment of resistance of each wall line. The torsional moment of resistance is determined by the design shear capacity of each wall line (used as the measure of relative stiffness) multiplied by the square of its distance from the center of stiffness. The amount of the torsional shear load (torsional moment) distributed to each wall line is then determined by the each wall's torsional moment of resistance in proportion to the total torsional moment of resistance of all shear wall lines combined. The torsional moment of resistance of each shear wall line and the total for all shear wall lines (torsional moment of inertia) is determined as shown below.

Wall Line	F_{psw}	Distance from Center of Resistance	$F_{psw}(d)^2$
PSW1	7,812 lb	19.3 ft	2.91×10^6 lb-ft^2
PSW2	3,046 lb	25.3 ft	1.95×10^6 lb-ft^2
PSW3	14,463 lb	12.1 ft	2.12×10^6 lb-ft^2
PSW4	9,453 lb	10.1 ft	9.64×10^5 lb-ft^2
PSW5	182 lb	44.7 ft	3.64×10^5 lb-ft^2
PSW6	9,453 lb	9.9 ft	9.26×10^5 lb-ft^2
PSW7	9,687 lb	22.7 ft	4.99×10^6 lb-ft^2
PSW8	11,015 lb	15.9 ft	2.78×10^6 lb-ft^2
Total torsional moment of inertia (*J*)			1.70×10^7 lb-ft^2

Now, the torsional shear load on each wall is determined using the following basic equation for torsion:

$$V_{WALL} = \frac{M_T d(F_{WALL})}{J}$$

where,

V_{WALL} = the torsional shear load on the wall line (lb)
M_T = the torsional moment* (lb-ft)
d = the distance of the wall from the center of stiffness (ft)
F_{WALL} = the design shear capacity of the segmented or perforated shear wall line (lb)
J = the torsional moment of inertia for the story (lb-ft^2)

*The torsional moment is determined by multiplying the design shear load on the story by the offset of the center of stiffness relative to the center of gravity perpendicular to the load direction under consideration. For wind design, the center of the vertical projected area of the building is used in lieu of the center gravity.

Now, the torsional loads may be determined as shown below for the N-S and E-W wall lines. For PSW1 and PSW2 the torsion load is in the reverse direction of the direct shear load on these walls. This behavior is the result of the center of shear resistance being offset from the force center which causes rotation about the center of stiffness. (Center of shear resistance and center of stiffness may be used interchangeably since the shear resistance is assumed to represent stiffness.) If the estimated offset of the center of gravity and the center of stiffness is reasonably correct, then the torsional response will tend to reduce the shear load on PSW1 and PSW2. However, codes generally do not allow the direct shear load on a wall line to be reduced due to torsion – only increases should be considered.

The following values for use in the torsion equation apply to this example:

M_T = (8,983 lb)(7 ft) = 62,881 ft-lb
J = 1.70 x 10^7 lb-ft^2

The torsional loads on PSW5 and PSW7 are determined as follows:

V_{psw5} = (62,881 ft-lb)(44.7 ft)(182 lb) / (1.70 x 10^7 lb-ft^2)
= 30 lb

V_{psw7} = (62,881 ft-lb)(22.7 ft)(9,687 lb) / (1.70 x 10^7 lb-ft^2)
= 813 lb

These torsional shear loads are added to the direct shear loads for the N-S walls and the total design shear load on each wall line may be compared to its design shear capacity as shown below.

N-S Wall Lines	Wall Design Capacity, F_{psw} (lb)	Direct Shear Load (lb)	Torsional Shear Load (lb)	Total Design Shear Load (lb)	Percent of Design Capacity Used
PSW1	7,812	3,387	na*	3,387	43% (ok)
PSW2	3,046	1,321	na*	1,321	43% (ok)
PSW5	182	81	30	111	61% (ok)
PSW7	9,687	4,195	813	5,008	52% (ok)

*The torsional shear load is actually in the reverse direction of the direct shear load for these walls, but it is not subtracted as required by code practice.

While all of the N-S shear wall lines have sufficient design capacity, it is noticeable that the wall lines on the left side (West) of the building are "working harder" and the walls on the right side (East) of the building are substantially over-designed. The wall construction could be changed to allow a greater sheathing nail spacing on walls PSW1 and PSW2. Also, the assumption of a rigid diaphragm over the entire expanse of the story is very questionable, even if the garage is "rigidly" tied to the house with adequate connections. It is likely that the loads on Walls PSW5 and PSW7 will be higher than predicted using the relative stiffness method. Thus, the tributary area method (see Step 2) may provide a more reliable design and should be considered along with the above analysis. Certainly, reducing the shear wall construction based on the above analysis is not recommended prior to "viewing" the design from the perspective of the tributary area approach. Similarly, the garage opening wall (PSW5) should not be assumed to be adequate simply based on the above analysis in view of the inherent assumptions of the relative stiffness method in the horizontal distribution of shear forces. For more compact buildings with continuous horizontal diaphragms extending over the entire area of each story, the method is less presumptive in nature. But, this qualitative observation is true of all of the force distribution methods demonstrated in this design example.

Conclusion

This seemingly simple design example has demonstrated the many decisions, variables, and assumptions to consider in designing the lateral resistance of a light-frame home. For an experienced designer, certain options or standardized solutions may become favored and developed for repeated use in similar conditions. Also, an experienced designer may be able to effectively design using simplified analytical methods (i.e. the total shear approach shown in Step 1) supplemented with judgment and detailed evaluations of certain portions or unique details as appropriate.

In this example, it appears that a 7/16-inch-thick Structural I wood structural panel sheathing can be used for all shear wall construction to resist the required wind shear loading. A constant sheathing panel edge nail spacing is also possible by using 3 inches on center if the perforated shear wall method is used and 4 inches on center if the segmented shear wall method is used (based on the worst-case condition of Wall Line B). The wall sheathing nails specified were 8d pneumatic nails with a 0.113 inch diameter. In general, this wall construction will be conservative for most wall lines on the first story of the example house. If the seismic shear load were the only factor (i.e., the wind load condition was substantially less than assumed), the wall construction could be simplified even more such that a perforated shear wall design approach with a single sheathing fastening requirement may be suitable for all shear wall lines. The garage opening wall would be the only exception.

Finally, numerous variations in construction detailing in a single project should be avoided as it may lead to confusion and error in the field. Fewer changes in assembly requirements, fewer parts, and fewer special details should all be as important to the design objectives as meeting the required design loads. When the final calculation is done (regardless of the complexity or simplicity of the analytic approach chosen and the associated uncertainties or assumptions), the designer should exercise judgment in making reasonable final adjustments to the design to achieve a practical, well-balanced design. As a critical final consideration, the designer should be confident that the various parts of the structural system are adequately “tied together” to act as a structural unit in resisting the lateral loads. This consideration is as much a matter of judgement as it is a matter of analysis.

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