PDHonline Course S203 (8 PDH)

Calculating and Designing Wood Framing Components for Light Frame Construction

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George E. Thomas, PE

1 General

This course will address the design of wood structural systems and construction materials commonly used is light-frame wood construction. The course focuses on structural design that specifies standard dimension lumber and structural wood panels (i.e., plywood and oriented strand board sheathing, etc.). Design of the lateral force resisting system (i.e., shearwalls and diaphragms) is approached from a system design perspective. The basic components and assemblies of conventional wood frame construction are shown in Figure 1.

Many elements of light frame construction 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. The issue of system performance is most pronounced in the above-grade assemblies of light-frame construction. Within the context of simple engineering approaches familiar to engineers, system-based design principles are addressed in this course.

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

- Floors
- Walls
- Roofs

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

- Bending members
- Columns
- Combined bending and axial loaded members;
- Sheathing (i.e., diaphragm)
- Connections

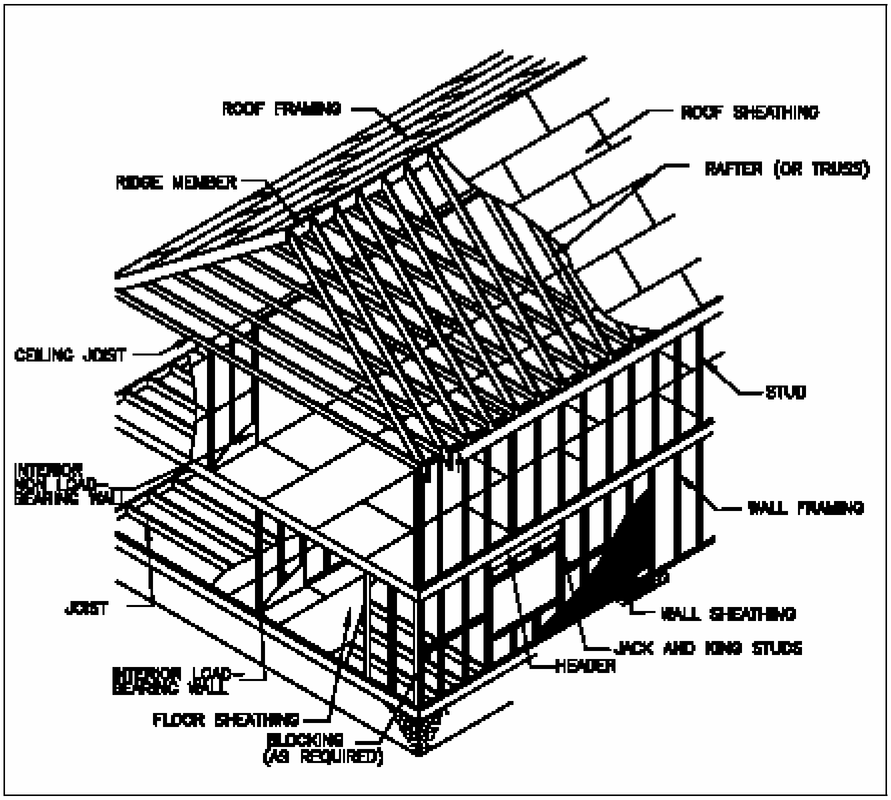
The principal method of design for wood-framed construction has been the allowable stress design (ASD). This course will be using the ASD method, however the load resistance factored design method (LRFD) is now available as an alternative. The engineer should obtain the National Design Specification (NDS) commentary to develop a better understanding of the rationale and substantiation for the National Design Specification and National Design Specification Supplement (NDS-S).

This course will look at 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 course focuses primarily on framing with traditional dimension lumber and will give some consideration to common engineered wood products. Other wood framing methods, such as post-and-beam construction, are not explicitly addressed, although much of the information is relevant. System considerations and system factors presented are only relevant to light, wood-framed construction using dimension lumber.

No matter what structural element is to be analyzed, the engineer must first determine nominal design loads. The loads that act on a framing member or system are usually calculated in accordance with the applicable provisions of the locally approved building code and past engineering practice.

While prescriptive design tables (i.e., span tables) and similar design aids commonly used in residential applications are not included herein, the engineer can save considerable design effort by consulting resources containing such information such as local, state, or national model building codes. Prescriptive design aids and framing practices can be found in various design and construction handbooks. 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*. The engineer should also obtain design data from a variety of proprietary engineered wood products that are suitable for many special design needs in residential construction. However, those materials generally should not be viewed as simple “one-to-one” substitutes for conventional wood framing and all special design and construction requirements must be carefully considered in accordance with the manufacturer’s recommendation or applicable code evaluation reports.

FIGURE 1 Components of a Conventional Wood-Framed Construction



2 Material Properties

It is essential that the engineer using wood materials know 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.

2.1 Lumber

General

The engineer must consider wood's strengths and weaknesses. Comprehensive technical information on wood characteristics can be found in the *Wood Engineering Handbook, Second Edition* (Forest Products Laboratory, 1990). The knowledge incorporated in this handbook is reflected in the provisions of the NDS and the NDS Supplement design data; , many aspects of wood design require good judgment by the engineer.

Wood is a natural material, as a structural material, demonstrates unique and complex characteristics. Wood's structural properties can be traced back to the material's natural composition. 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 and is referred to as the slope of grain.

Wood Species

Structural lumber can be manufactured from a variety of wood species; 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* 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 and various grading organizations. 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 design and construction 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" difference between the nominal and dressed sizes of "dry" sawn lumber. For example, a 2x4 is actually 1.5" by 3.5", a 2x10 is 1.5" by 9.25", and a 1x4 is 1/2" by 3.5". This course uses nominal member size, it is important to note that the engineer must apply the actual dimensions of the lumber when analyzing structural performance or detailing construction dimensions.

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

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

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

Lumber Grades

Lumber is graded in accordance with standardized grading rules which consider the effect of natural growth characteristics and "defects," Mainly knots and slope 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 individuals who examine the wood members at the mill in accordance with a approved agencies grading rules.
The grader identifies wood members that are then separated into the appropriate grade classes. Typical visual grading classes are Select Structural, No. 1, No. 2, Stud, etc. Refer to the NDS Supplement or a national grading agency for more information on grades of different species of lumber. The engineer should consult local lumber suppliers or contractors regarding 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 may vary among grading agencies, the U.S. Department of Commerce has set forth minimums for voluntary adoption by the recognized lumber grading agencies. For more information regarding grading rules, refer to the National Institute for Standards and Technology.

Moisture Content

Wood properties and dimensions change with moisture content (MC). Wood contains varying amounts 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 around 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 9 percent in a protected environment, it should be dried or seasoned before installation. Proper drying and storage of lumber minimizes 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 maximum moisture content of 15 percent. The tabulated design values are based on moisture content of 19 percent for dimension lumber.

Engineers need to plan for 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 engineer might have to account for differential shrinkage by isolating members that will shrink from those that will maintain dimensional stability. The engineer should detail the structure so that shrinkage is as uniform as possible, thereby minimizing shrinkage effects on finish surfaces. Details minimizing the amount of wood transferring loads perpendicular to slope of grain are preferable.

Shrink and swell can be calculated in accordance with Section 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 the 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 high moisture levels 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
- Specifying preservative-treated or naturally decay-resistant wood

An exterior weather barrier (e.g., roofing and siding) protects most structural wood, although improper design 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.

Wood members that are in ground contact should be preservative treated. Check the American Wood-Preservers' Association (AWPA) standards for types of treatments used for applications such as sill plates located near the ground or for exterior decks. It is important to specify the correct type and level of treatment.

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 construction require periodic inspection for signs of termites.

2.2 Structural Wood Panels

In past construction boards have been used for roof, floor, and wall sheathing; today, structural wood panel (plywood, OSB, etc.) products are dominating the sheathing market. Structural wood panel products are more economical and efficient and are considered to be stronger than traditional board sheathing.

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. This allows the plywood to be stronger in bending along the long direction and therefore should be placed with the long dimension spanning floor and roof framing members. The number of plies 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 and is used for the same applications as plywood.

The engineer 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 4'x 8', with thicknesses typically ranging from 3/8" to over 1". Panels can be ordered in varying lengths for all types of applications and are stamped with their rating.

Plywood is performance-rated according to the provisions of U.S. Department of Commerce (USDOC) PS-1 for industrial and construction plywood. OSB products are performance-rated according to the provisions of USDOC PS-2. 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 or 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 differ in moisture resistance. 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

Most 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). Although the second rating method does not specifically indicate wall stud spacing, the panels may also be used for wall sheathing. The APA design and construction guide for residential and commercial construction provides a correlation between roof/floor ratings and allowable wall support spacing. The Load-Span Tables for APA Structural-Use Panels provides span ratings for various standard and nonstandard loading conditions and deflection limits.

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

Characteristic structural properties for use in ASD and load and LRFD are used to establish design values. Test data collected in accordance with applicable standards determine a characteristic strength value for each grade and species of lumber. These values are 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 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 factor 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 2.4.

- 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)
- $F_{c\perp}$ reduction factor = (2/3 end position factor)

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. 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 2 presents adjustment factors that apply to different structural properties of wood. The following sections will 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 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	1/2.1	Fifth percentile	Ultimate Capacity	D1990
Tension parallel to grain, F_I	1/2.1	Fifth percentile	Ultimate Capacity	D1990
Shear parallel to grain, F_v	1/4.1	Fifth percentile	Ultimate Capacity	D245
Compression parallel to grain, F_c	1/1.9	Fifth percentile	Ultimate Capacity	D1990
Compression perpendicular to grain, F_{c1}	1/1.5	Mean	0.04" Deflection	D245
Modulus of elasticity, E	1/1.0	Mean	Proportional Limit	D1990

Notes:
The characteristic design value for F_{c1} is controlled by a deformation limit state. The lumber will densify and carry an increasing load as it is compressed.
The proportional limit of wood load-deformation behavior is not clearly defined because it is nonlinear. Designation of a proportional limit is subject to variations in interpretation of test data.

TABLE 2 Adjustment Factor Applicability to Design Values for Wood

Design Properties	**Adjustment Factor**														
	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							✔			✔		✔	✔		

Notes:
Basic or unadjusted values for design properties of wood are found in NDS-S. See Table 1 for definitions of design properties.
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 2.4.1 and refer to 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 2.4.2 and refer to 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 2.4.3 and refer to 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 2.4.4 and refer to NDS-S).
- C_P, Column Stability Factor. Applies to lateral support condition of compression members (see Section 2.4.5 and refer to 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 2.4.6 and refer to NDS 3.3.3).
- C_M, Wet Service Factor. Applies where the moisture content is expected to exceed 19 percent for extended periods (refer to 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 (refer to 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 (refer to 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 refer to 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 (refer to 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 refer to 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 (refer to NDS 2.3.11).
- C_c, Curvature Factor. Applies only to curved portions of glued laminated bending members (refer to NDS 5.3.4).
- C_f, Form Factor. Applies where bending members are either round or square with diagonal loading (refer to NDS 2.3.8).

2.4.1 Load Duration Factor (*CD*)

Lumber strength is affected by the cumulative duration of maximum variable loads experienced during the life of the structure. 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 approximately twice as much stress as the standard 10year 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 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 3 Recommended Load Duration Factors for ASD

Permanent (dead load)	Lifetime	0.9
Normal	Ten years	1.0
Occupancy (live load)	Ten years to seven days	1.0 to 1.25
Snow	One month to seven days	1.15 to 1.25
Temporary construction	Seven days	1.25
Wind and seismic	Ten minutes to one minute	1.6 to 1.8
Impact	One second	2.0

Notes:
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.
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.
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.

2.4.2 Repetitive Member Factor (C_r)

When three or more parallel dimension lumber members are spaced a maximum of 24" 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 residential 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; an increase in allowable stress is permitted in order to account for repetitive members. While the NDS recommends a repetitive member factor of 1.15 or a 15% increase in bending strength, system assembly tests have demonstrated that the NDS repetitive member factor is conservative for certain conditions. In fact, test results from several studies support the range of repetitive member factors shown in Table 4 for certain design applications. As shown in Table 2, the adjustment factor applies only to extreme fiber in bending, F_b. Later sections in this course cover other system adjustments related to concentrated loads, header framing assemblies, and deflection (stiffness) considerations.

TABLE 4 Recommended Repetitive Member Factors for Dimension Lumber Used in Framing Systems

Application	Recommended C_r Value	References
Two adjacent members sharing	load1.1 to 1.2	AF&PA,1996b HUD, 1999
Three adjacent members sharing load	1.2 to 1.3	ASAE, 1997
Four or more adjacent members sharing load	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.)	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	1.5–2x4 or smaller 1.35–2x6 1.25–2x8 1.2–2x10	AF&PA, 1996b SBCCI, 1999 Polensek, 1975

Notes:
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.
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.
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 noncontinuous 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.
Refer to NDS 4.3.4 and the NDS *Commentary* for additional guidance on the use of the 1.15 repetitive member factor.
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).

Values in Table 4 are provided for use by the engineer as an recommended "alternative" from the NDS. 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-FrameStructures (FPRS, 1983).

2.4.3 Horizontal Shear Factor (*CH*)

Because 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 apply when repetitive framing or built-up members are used. For members with no splits C_H equals 2.0.

Future allowable horizontal shear values will be increased by a factor of 2 or more because of changes 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 2.3. As an interim solution, a shear adjustment factor, C_H, of 2.0 should be apply to all designs that use horizontal shear values in 1997 and earlier editions of the NDS. As shown in Table 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.

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" for No. 3 or better, 6" 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 2 indicates the tabulated values that should be adjusted to account for size differences. The adjustment applies to visually graded lumber is 2" to 4" thick or when a minimum 5" thick rectangular bending member exceeds 12" in depth. Refer to NDS-S for the appropriate size adjustment factor.

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 nonsymmetric 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 2. Refer to the NDS for the equations used to calculate the column stability factor.

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 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 engineer 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; the lesser of these factors applies. Refer to the NDS 3.3.3 for the equations used to calculate C_L.

3 Structural Evaluation

As with all structural design, the engineer should perform several checks with respect to various design factors. This section provides an overview of checks and design concerns for the engineer. 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 course will address those design checks and provide examples of different structural systems and elements in residential construction. In addition, this course will provide instruction in the efficient design of light framed construction, the engineer should referred to the NDS for symbol definitions, as well as other guidance.

3.1 Structural Safety Checks

Bending (Flexural) Capacity

The following equations determine if a member has sufficient bending strength. Notches in bending members should be avoided, but small notches are permissible. 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.

Bending Equations (NDS 3.3)

$f_b \le F_b'$	basic design check for bending stress
$F_b' = F_b$ x	(applicable adjustment factors per Section 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

Shear parallel to grain (i.e., horizontal shear) is induced by bending action. It is 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. 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 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. Also, bolted connections influence the shear capacity of a beam. 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.

Horizontal Shear Equations (NDS 3.4)

$f_v \le F'_v$	basic design check for horizontal shear
$F'_v = F_v$ x	(applicable adjustment factors per Section 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"is recommended. For bending members bearing on masonry, a minimum bearing of 3 inches is advised. The resulting bearing areas may not 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 are based on net bearing area. Note that the NDS provisions acknowledge that the inner bearing edge experiences added pressure as the member bends. The added pressure does not pose a problem because the compressive capacity, $F'_{c\perp}$, of wood increases as the material is compressed. The design value is based on a deformation limit, not on failure by crushing. Therefore, the added pressure at bearing edges need not be considered. The engineer should use 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.

Bearing Equations (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 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 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 engineer 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 due to the design being limited by stability rather than by a stress failure mode. Efficiency issues will become evident when the engineer calculates the components of the combined stress interaction equations that are given below.

Combined Bending and Axial Loading Equations (NDS 3.9)

Combined bending and axial tension design check

$$\frac{f_t}{F_t'}+\frac{f_b}{F_b^*}\leq 1$$

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

Combined bending and axial compression design check

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

Compression and Column Stability

For framing members that support axial loads only (i.e., columns), the engineer 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 engineer should increase member size, decrease 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 engineer performs the calculations necessary to determine the stability factor, C_P.
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.

Compression and Column Stability Equations (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 2.4, including C_p)

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

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

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

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

Tension

Few members in light-frame construction resist tension forces only. One exception occurs in roof framing where cross-ties or bottom chords in trusses primarily resist tension forces. Other examples include chord and collector members in shear walls and horizontal diaphragms. Another possibility is a member subject to excessive uplift loads such as those produced by extreme wind. Connection design is usually the limiting factor in designing the transfer of tension forces in light-frame construction. Tension stresses in wood members are checked by using the equations below.

Tension Equations (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 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

In evaluating cross-grain tension forces 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 engineer 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.

3.2 Structural Serviceability

Deflection Due to Bending

The engineer must defer to engineering judgment and building code specifications for limits in deflection. With many interior and exterior finishes susceptible to damage by large deflections, reasonable deflection limits based on design loads are recommended 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.

Deflection Due to Bending Equations (NDS 3.5)

$$\Delta_{estimate} \leq \Delta_{allow} = \frac{\ell}{(120 \text{ to } 600)} \quad \text{(see Table 5 for value of denominator)}$$

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

If a deflection check proves unacceptable, the engineer 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 the application and engineers judgment. Table 5 provides recommended deflection limits. If a modest adjustment to a deflection limit results in a more efficient design, the engineer 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.

TABLE 5 Recommended Allowable Deflection Limits

Element or Condition	Deflection Limit, Δ_{all}	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	$\ell/360$	L
Floor girders and beams	$\ell/360$	L

Notes:
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.
l is the clear span in units of inches for deflection calculations.
Floor vibration may be controlled by using l/360 for spans up to 15 feet and a 1/2-inch limit for spans greater than 15 feet. Wood I-joist manufacturers typically recommend l/480 as a deflection limit to provide enhanced floor performance and to control nuisance vibrations.
Floor vibration may be controlled for combined girder and joist spans of greater than 20 feet by use of a l/480 to l/600 deflection limit for the girder.

System effects influence the stiffness of assemblies in a manner similar to that of bending capacity (see Section 2.4.2), the system deflection factors of Table 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. Deflection checks on floors under uniform loading can be easily overestimated by 20% or more. In areas where partitions add to the rigidity of the supporting floor, deflection can be overestimated by more than 50%. 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. Similar results have been found for sheathed wall assemblies. When adhesives attach wood structural panels to wood framing, even greater reductions in deflection are realized due to increased composite action. However, if a simple deflection limit such as l/360 is construed to control floor vibration in addition to the serviceability of finishes, the use of system deflection factors of Table 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 6 System Deflection Adjustment Factors

Framing System	Multiply single member deflection estimate by:
Light-wood-frame floor system with minimum 2x8 joists, minimum 3/4-inch-thick sheathing, 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	0.7–2x4 0.8–2x6

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

Floor Vibration

Reliable design information on controlling floor vibration to meet a specific level of “acceptance” is not readily available; The following rules of thumb are provided for the engineer wishing to limit vibration beyond that implied by the traditional use of an l/360 deflection limit.

- For floor joist spans less than 15 feet, a deflection limit of l/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 l/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 l/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 are 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 calculated by the equation below for typical softwood structural lumber. Shrinkage in the longitudinal direction of the member is practically negligible.

Shrinkage Equation (ASTM 1990)

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

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)

4 Floor Framing

The objectives of floor system design are:

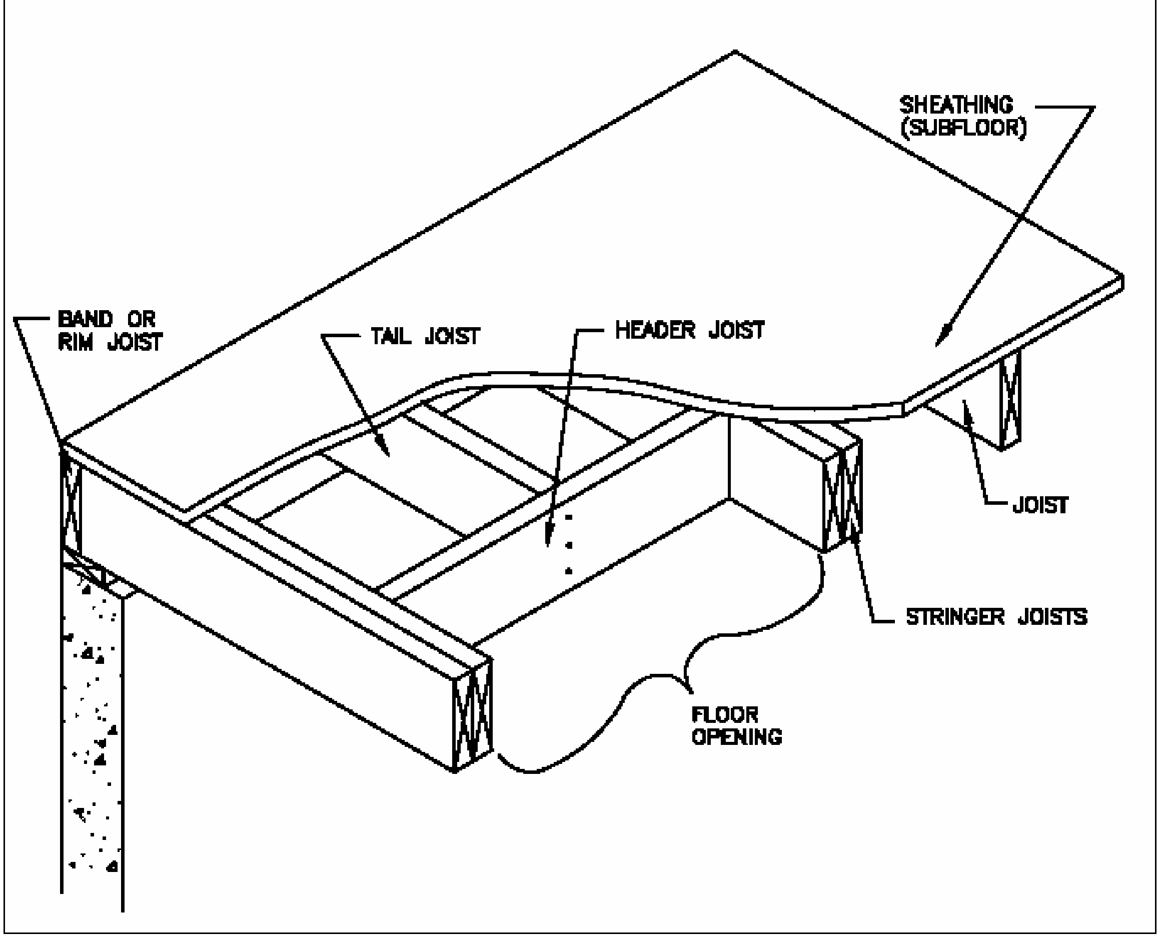
- To support occupancy live loads and building dead loads adequately;
- To resist lateral forces resulting from wind and seismic loads and to transmit the forces to supporting shear walls through diaphragm action;
- To provide a suitable subsurface for floor finishes;

- 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).

4.1 General

Wood floor systems include joists, girders, and sheathing. Traditionally these systems have been built of solid sawn lumber, parallel chord wood trusses and wood I-joists are now being 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. See Figure 2 for an illustration of floor system structural elements.

FIGURE 2 Structural Elements of the Floor System



The design approach in this course 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 manufacturer's specifications and applicable code evaluation reports.

Section 3 discusses the general design equations and design checks. The present section provides detailed design examples that apply the equations in Section 3, while tailoring them to the design of the elements in a floor system. The following sections make reference to the span of a member (for this course span is defined as the clear span between bearing points).

When designing any structural element, the engineer must first determine the loads acting on the element. Load combinations used in this course are listed below.

Load Combinations used for the design of Components and Systems

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

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

Given that only the dead loads of the floor system and live loads of occupancy are present in a typical floor system, the controlling design load combination for a simply-supported floor joist is D+L. For joists with more complicated loading, such as cantilevered joists supporting roof framing, the following load combinations may be considered.

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

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

4.2 Floor Joist Design

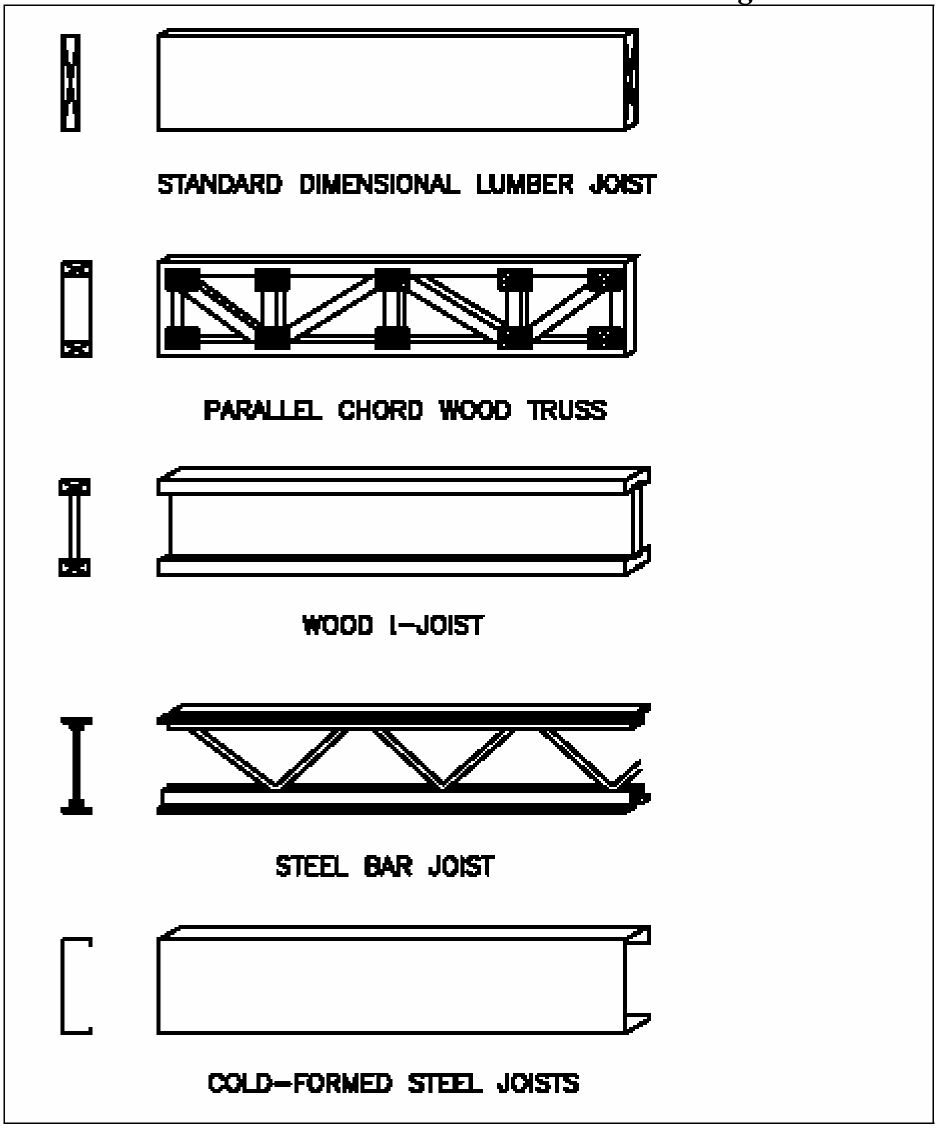
Readily available tables in residential building codes provide maximum allowable spans for different species, grades, sizes, and spacing of lumber joists. Therefore, it is usually not necessary to design conventional floor joists for residential construction. To obtain greater economy or performance engineers may want to create their own span tables or spreadsheets for future use in accordance with the methods shown in this course.

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 (HF), Spruce-Pine-Fir (SPF), Douglas-Fir (DF), and Southern Yellow Pine (SYP). The most common sizes for floor joists are 2x8 and 2x10, although 2x12s are also frequently used. The design examples provided in this course illustrate the design of typical floor joists in accordance with the principles discussed earlier:

- simple span joist (Examples 1 and 2); and
- cantilevered joist (Example 3).

For different joist applications, such as a continuous multiple span, the engineer should use the appropriate beam equations (see 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 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), see Section 6.3 on roof trusses. Cold-formed steel floor joists or trusses may also be considered. Figure 3 illustrates some conventional and alternative floor joist members.

FIGURE 3 Conventional and Alternative Floor Framing Members



Notes:
Trusses are also available with trimmable ends. 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. 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 abovementioned design methodology for a single, concentrated load applied near midspan of a repetitive member floor system is essentially equivalent to using a C_r factor of 1.5 or more (see Section 2.4.2). The system deflection adjustment factors in Table 6 are applicable as indicated for concentrated loads.

Bridging or cross-braces is thought to provide necessary lateral torsional bracing of dimension lumber floor joists and stiffer floor systems. However, testing of different floor systems of residential structures has conclusively demonstrated that bridging or cross-bracing provides little 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. These 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" in depth. 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). However, most codes require bridging to be spaced at intervals not exceeding 8 feet along the span of 2x10 and 2x12 joists. Engineering judgment and local codes should be considered when requiring bridging and cross-bracing of floors.

4.3 Girder Design

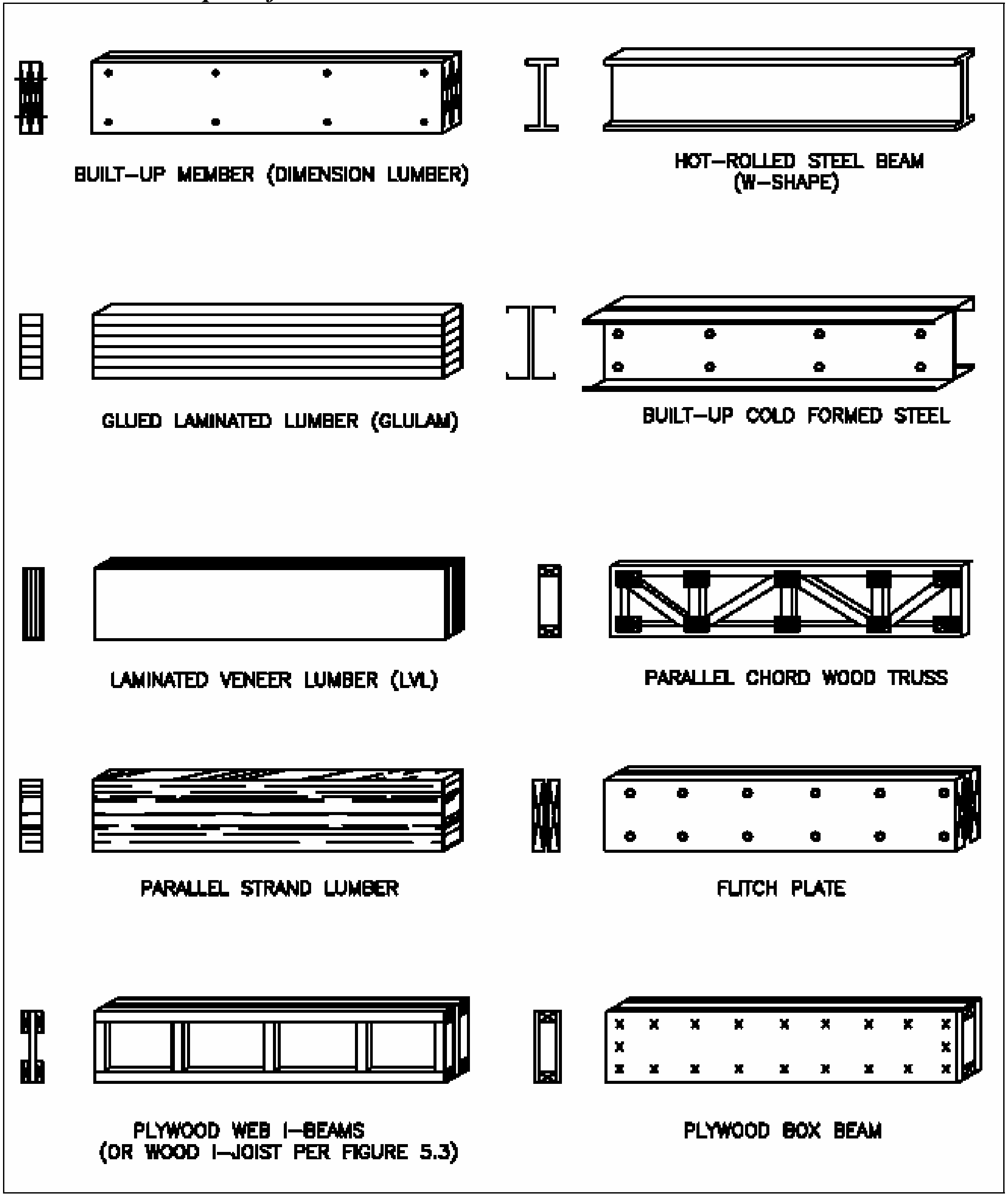
The decision to use one girder over another is a function of cost, availability, span and loading conditions, clearance or head-room requirements, and ease of construction. Figure 4 illustrates girder types. Girders in residential construction are usually one of the following types:

- Built-up dimension lumber
- Steel I-beam
- Engineered wood beam
- Site-fabricated beam
- Wood I-joist
- Metal plate connected wood truss

Built-up beams are constructed by nailing together 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" on center top to bottom. The design method and equations are the same as those in Section 4.2 for floor joists; however, the adjustment factors applying to design values and loading conditions are different. The engineer shall 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
- Girders supporting floor framing, lateral support is considered to be continuous and $C_L = 1$. Example 4 illustrates the design of a built-up floor girder.

FIGURE 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. Steel beam shapes are commonly referred to as Ibeams, however 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* 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.

Structural steel floor beam span tables are also found in the *Beam Series*. The *Prescriptive Method for Cold-Formed Steel in Residential Construction* should be consulted for the design of built-up coldformed steel sections as headers and girders.

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

Site-fabricated beams include plywood box beams, plywood I-beams, and flitch plate beams. *Plywood box beams* are fabricated from continuous 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" 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" thick and about 1/4" less in depth than the dimension lumber. The sandwich construction is usually assembled with through-bolts staggered at about 12" 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 Ibeams, and flitch plate beams are provided in NAHB's *Beam Series* publications. Refer to the APA's *Product Design Specification* and *Supplement* for the design method used for plywood box beams. The *International One- and Two-Family Construction Code*, provides a simple prescriptive table for plywood box beam headers.

4.4 Subfloor Design

Typical subfloor sheathing is nominal 5/8"or 3/4" 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 uses the *Design and Construction Guide: Residential and Commercial* 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 engineer 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.

The APA also recommends a fastener schedule for connecting sheathing to floor joists. Generally, nails are placed a minimum of 6"on center at edges and 12"on center along intermediate supports. See Table 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. 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 and should be determine by the engineer. Regardless of fastener type, gluing the floor sheathing to the joists increases floor stiffness and strength.

TABLE 7 Fastening Floor Sheathing to Structural Members

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:
Codes generally require common or box nails; if pneumatic nails are used, as is common, refer to the nail manufacturer's data. Screws are also commonly substituted for nails.

Boards may also be used as a subfloor (i.e., board sheathing). Floor sheathing boards are typically 1x6 or 1x8 material laid flat and diagonally (or perpendicular) on the floor joists. They should be designed using accepted engineering practice.

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.

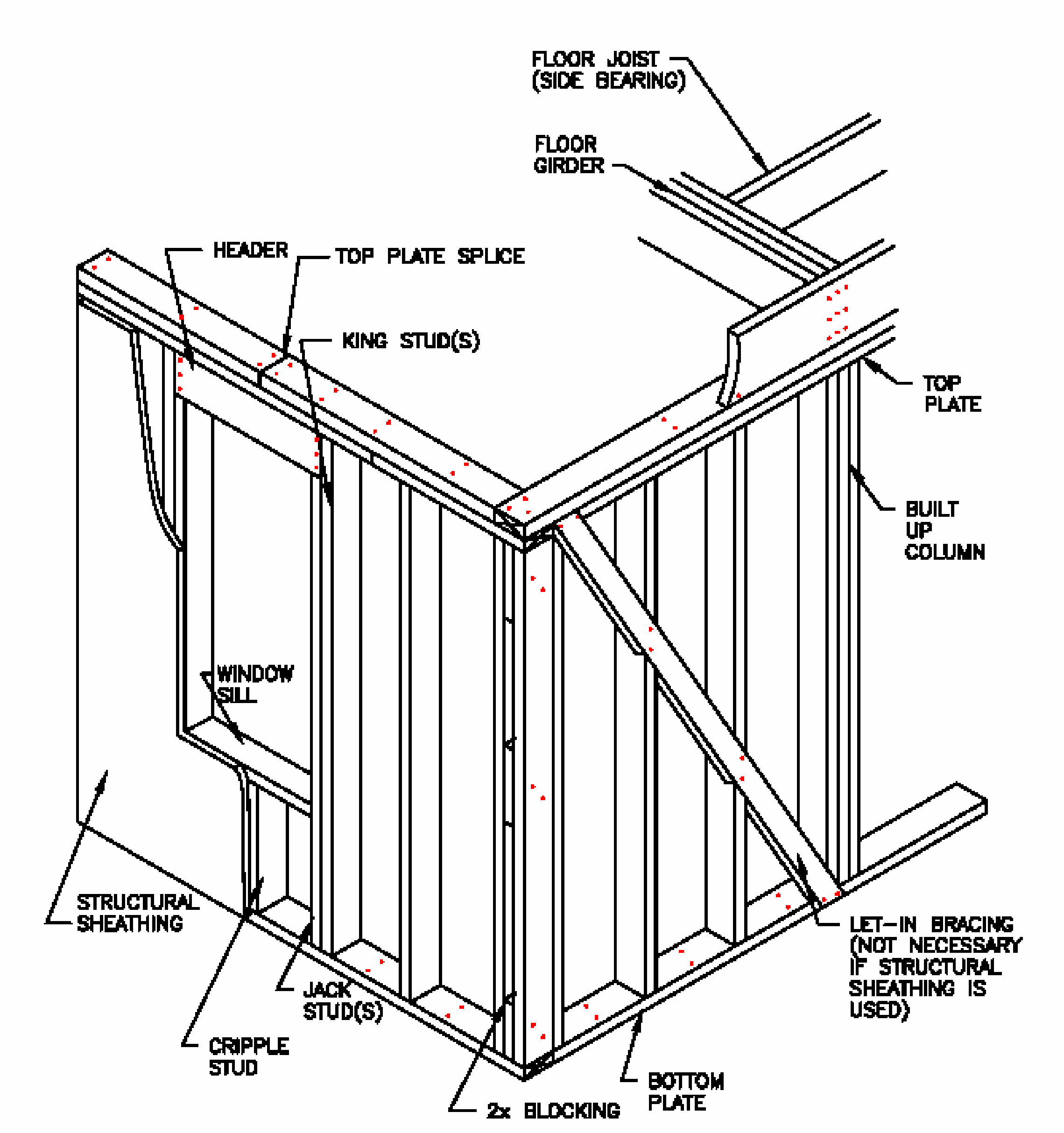
5.1 General

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 and it will resist lateral loads resulting from wind and earthquakes. A typical wood-framed wall is composed of the following elements as shown in Figure 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, engineered wood studs and cold-formed steel studs are now being use as well. 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 and by other names) 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 Structural Elements of the Wall System



Structural wall sheathing (plywood or oriented strand board, etc.) 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 residential construction now use wood structural panel braces, and many of those homes are fully sheathed with wood structural panels. These bracing methods are substantially stronger than the let-in brace approach. Wood let-in braces are typically 1x4 wood members that are "let-in" or notched into the studs and nailed diagonally across wall sections at corners and specified intervals. Their use is generally through application of conventional construction provisions found in most building codes for residential construction in combination with interior and exterior claddings.

The design procedures, in this course, addresses dimension lumber wall systems and provides standard design equations and design checks. Detailed design examples 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 engineer uses the "system strength" inherent in the construction. Recommendations on system design are in Sections 2 and 3.

When designing wall elements, the engineer needs to consider the load combinations, particularly dead, live, snow, and wind loads:

1. $D + L + 0.3\ (L_r \text{ or } S)$
2. $D + (L_r \text{ or } S) + 0.3\ L$
3. $D + W$
4. $D + 0.7E + 0.5L + 0.2S$

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 and 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 engineer should consider out-of-plane bending loads resulting from an earthquake load combination, although the other load combinations above usually control the design. The third and fourth load combinations are essentially combined bending and axial loads that may govern stud design as opposed to axial load only in the first two load combinations.

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

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 structures. Example 6 demonstrates the design of an exterior bearing wall.

When calculating the column stability factor for a stud wall column capacity is determined by using the slenderness ratio about the strong axis of the stud $(l_e/d)_x$. The reason for using the strong axis slenderness ratio is that lateral support is provided to the stud by the wall sheathing and finish materials in the stud's weak-axis bending or buckling direction. When determining the column stability factor, C_p, for a wall system rather than for a single column, the engineer 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 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 engineer 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 5.3). 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 to the outside walls to provide for attachment of interior walls and installation of plywood, OSB, and gypsum wallboard. 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 engineer may follow the design procedure demonstrated in Example 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, it is recommended that the engineer design load-bearing walls in accordance with the previous requirements for 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 2.4.1.)

5.3 NonLoad-Bearing Partitions

Interior partitions are not a support for structural loads. Standard 2x4 or 2x3 wood stud interior partition walls are well proven in standard 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 2x3 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 2 and 3 apply as appropriate.

5.4 Headers

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

Load-bearing header design and fabrication is similar to that for girders (see Section 4.3). Headers consisting of double members to be considered repetitive members; a repetitive member factor, C_r, of 1.1 to 1.2 should apply (see Table 4), along with a live load deflection limit of $l/240$ (see Table 6). Large openings or especially heavy loads may require stronger members such as engineered wood beams, hot-rolled steel, or flitch plate beams.

Headers are generally designed to support all loads from above, residential construction requires 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. Tests have shown that a system factor or repetitive member factor is valid for headers constructed of only two members as shown in Table 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 7. An overall system factor of 1.8 is 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.

Table 8 are recommended allowable bending stress adjustment factors for use in the specific header design conditions related to the discussion above. For other conditions, see Table 4. Example 7 demonstrates the design approach for a typical header condition.

TABLE 8 Recommended System Adjustment Factors for Header Design

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

Notes:
For other applications and lumber sizes or grades, refer to the C_r factors in Table 4 of Section 2.4.2.
Apply C_r in lieu of Section 3 (Table 4) to determine adjusted allowable bending stress, F_b'.
Use C_r = 1.35 when the header is overlaid by a minimum 2x4 double top plate without splices.
Refer to Example 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 some builders use additional jamb studs for openings in nonload-bearing walls, however such studs are not required.

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 3 account for the slenderness factor.

Figure 6 illustrates three ways to construct columns using lumber. *Simple columns* are columns fabricated from a single piece of sawn lumber; *spaced columns* are fabricated from two or more individual members with their longitudinal axes parallel and separated with blocking at their ends and midpoint; *built-up columns* are solid columns fabricated from several individual members fastened together. Spaced columns are not normally used in residential structures and have not been addressed here.

Steel jack posts are also commonly used in residential construction. Jack post manufacturers typically provide a rated capacity, therefore designing steel jack posts are not required except for the specification of the design load requirements and the selection of a suitable steel 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.

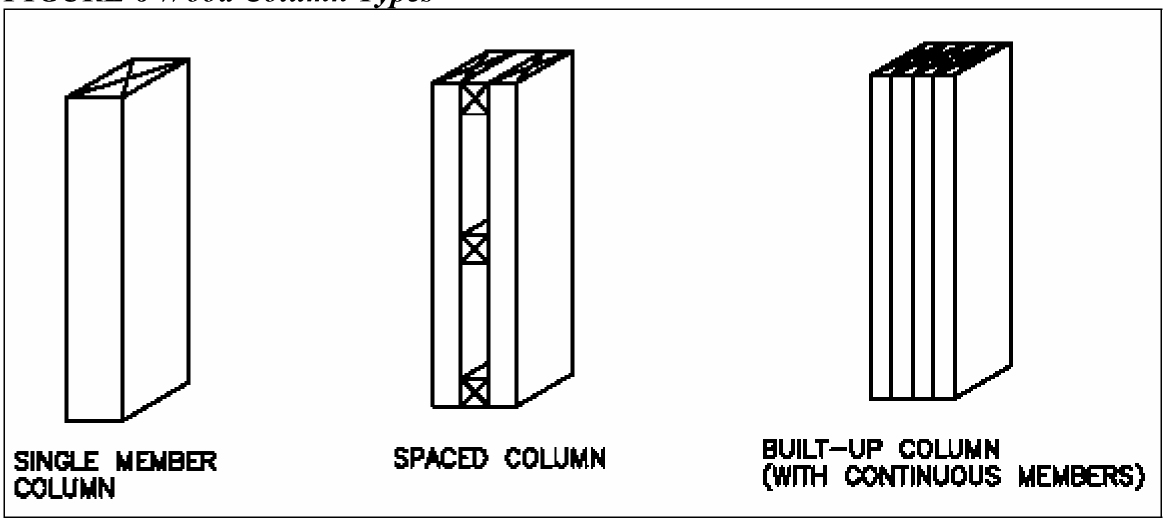
Simple columns are fabricated from one piece of sawn lumber. In residential construction, simple columns such as a 4x4 are common. The equations in Section 3 are used to design simple columns as demonstrated in Example 8.

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; 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 only to the weak-axis buckling or bending direction of the individual members and should not be used to determine C_p for column buckling in the strong-axis direction of the individual members.

The above 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 4). However, when the members in the built-up column are staggered or spliced, the column bending strength is reduced. Design methods and nailing requirements for spliced columns are ready available.

FIGURE 6 Wood Column Types



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
- To provide attic space and access for electrical and mechanical equipment or storage

6.1 General

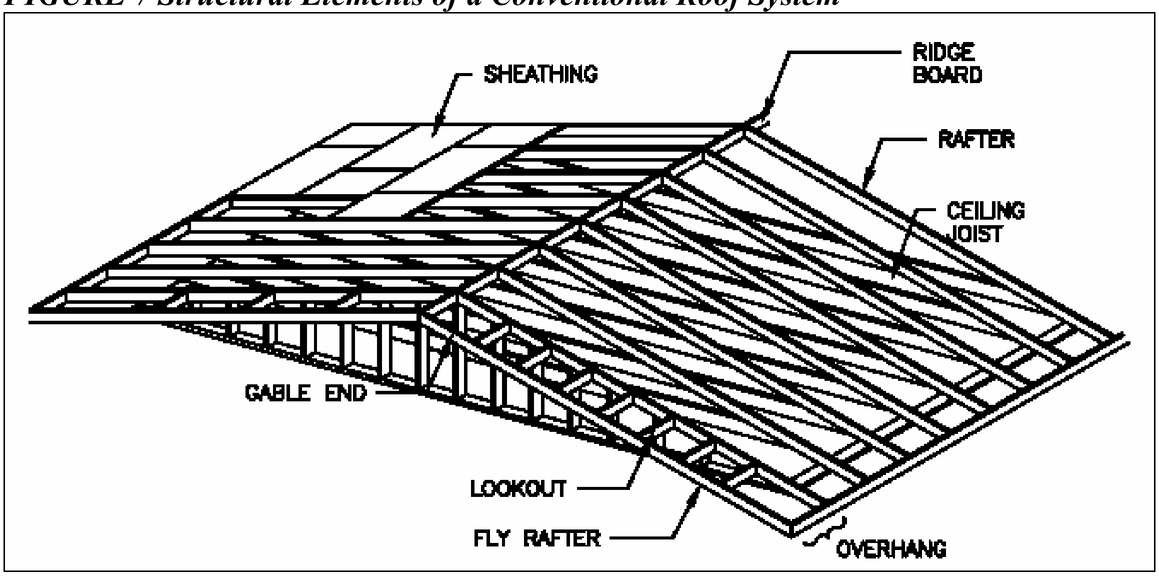
A roof in a residential structure is typically a sloped structural system that supports gravity and lateral loads and transfers the loads to the walls below. Generally, the four options for wood roof construction are

- Roof trusses
- Rafters and cross-ties
- Rafters with ridge beams (i.e. cathedral ceiling); and
- Timber framing

Most residential roof construction use light-frame trusses, rafters, or a mix of these depending on roof layout. Figure 7 shows 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, they are grouped together them under one section.

FIGURE 7 Structural Elements of a Conventional Roof System



Roof trusses are pre-engineered components. They are constructed from 2” thick dimension lumber connected with metal truss plates. They are 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 structural element, usually plywood or oriented strand board, that supports roof loads and distributes lateral and axial loads to the roof framing system. Roof sheathing also provides lateral support to the roof framing members and serves as a membrane or diaphragm to resist and distribute lateral building loads from wind or earthquakes.

Roof systems are designed to withstand dead, live, snow, and wind uplift loads and are designed to withstand lateral loads, such as wind and earthquake loads, transverse to the roof system. The design procedure discussed here address dimension lumber roof systems. Section 3 summarizes the general design equations and design checks of roof systems.

When designing roof elements or components, the engineer needs to consider the following load combinations:

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

The following refers to the span of the member. A span of a member is 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 in this course.

Roofs exhibit system behavior that is in many respects similar to floor framing (see Section 4); 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. Research on trussed roof assemblies with wood structural panel sheathing has show a system capacity increase factor of 1.1 to 1.5 relative to the design of an individual truss. A conservative system factor of 1.15 is used in this course and is to be considered by the engineer for chord bending stresses and a factor of 1.1 is to be considered for chord tension and compression stresses.

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 are built in place at the site. 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 sized according to their individual applied bending loads taking into account that the axial load effects on the members themselves can be dismissed by the engineers judgment based on the large system effects in sheathed roof construction. Frequently, intermediate rafter braces that are similar to truss web members are also used. Standard construction details and span tables for rafters and ceiling joists can be found in the *International One- and Two-Family Dwelling Code*. These tables generally provide allowable horizontal rafter span with disregard to any difference that roof slope may have on axial and bending loads experienced in the rafters.

This approach is generally considered as standard practice. Example 9 demonstrates two design approaches for a simply-supported, sloped rafter as illustrated in Figure 8.

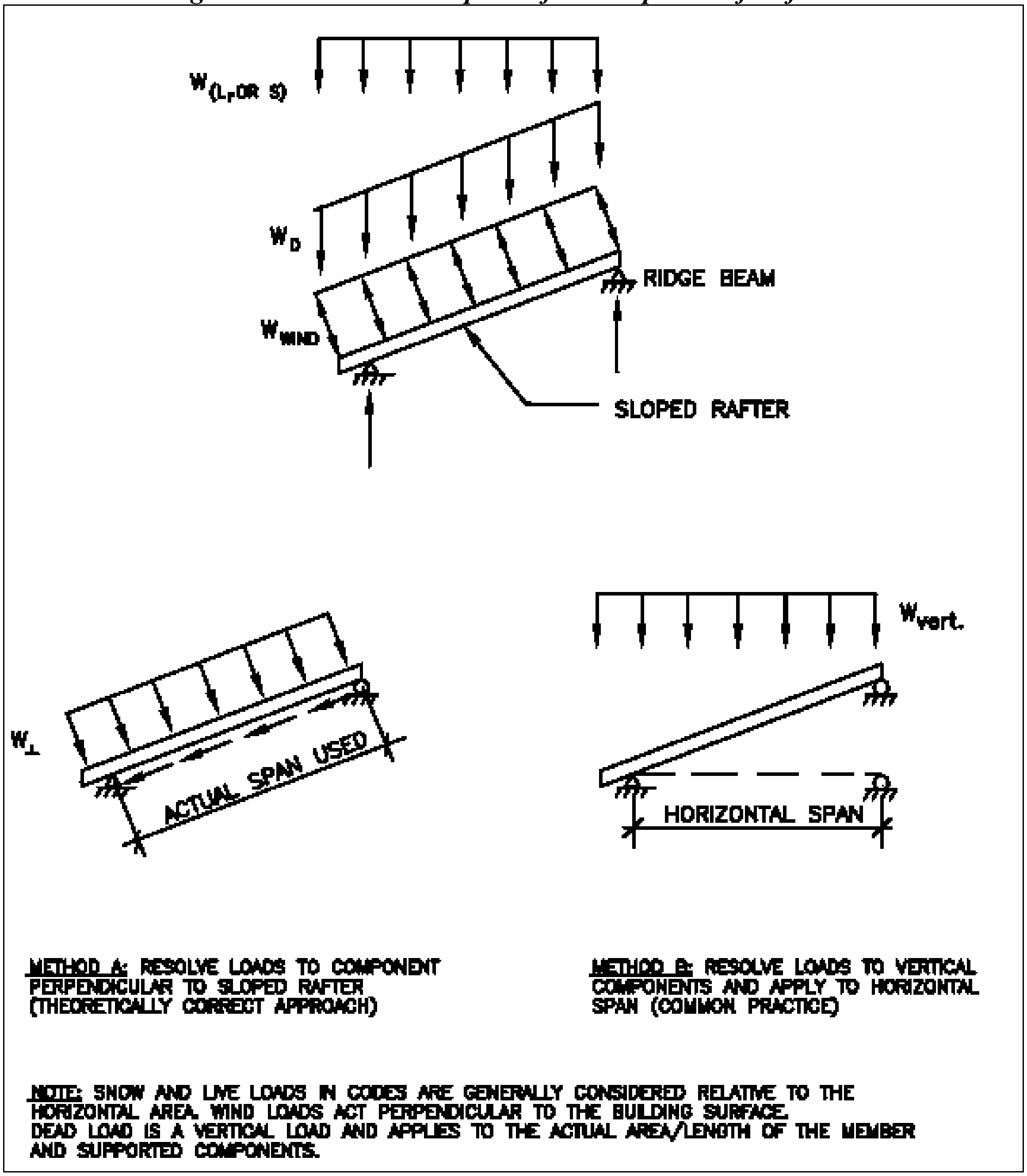
Structural ridge beams are designed to support roof rafters at the ridge when there are no ceiling joists or cross-ties to resist the outward thrust of rafters that would otherwise occur. A repetitive member factor, C_r, is applicable if the ridge beam is composed of two or more members (see Table 4). It should 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 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. Beneficial system effect generally goes ignored because of the lack of definitive technical guidance. The use of engineering judgment should not be ruled out. Example 11 demonstrates the design of a hip rafter.

6.3 Roof Trusses

Roof trusses incorporate rafters (top chords) and ceiling joists (bottom chords) into a structural frame fabricated from 2” 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 structure 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.

FIGURE 8 Design Methods and Assumptions for a Sloped Roof Rafter



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

- Design loads
- Truss profile
- Support locations
- Any special requirements

The engineer 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 end wall bracing is discussed separately in Section 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*. Temporary bracing during construction is usually the responsibility of the contractor and is important for worker safety.

The National Design Standard for Metal Plate Connected Wood Truss Construction governs the design of trusses. Available from the Truss Plate Institute includes the structural design procedure as well as requirements for truss installation and bracing and standards for the manufacture of metal plate connectors. Computer programs are also available for a detailed finite element analysis. Truss plate manufacturers and truss fabricators generally have proprietary computerized design software tailored to their particular truss-plate characteristics.

The engineer 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, buried under attic insulation, may not be similarly affected. This may bow the truss 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” 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.

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. Trusses have been used to provide a complete residential frame. 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.

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 manufacturers literature. If the limit of the prescriptive tables is exceeded, the engineer may need to perform calculations; however, such calculations are rarely necessary for residential structures.

The process of selecting rated roof sheathing is similar to that for floor sheathing in Example 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. 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" on center at intermediate supports. A 6" 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" thick on residential roofs. Note that in some cases shear loads in the roof diaphragm resulting from lateral loads (i.e., wind and earthquake) may require a more stringent fastening schedule. More importantly, large suction pressures on roof sheathing in high wind areas will require a larger fastener and/or closer spacing. In hurricane-prone regions, it is common to require an 8d deformed shank nail with a 6" on center spacing at all framing connections. At the gable end truss or rafter, a 4" spacing is common.

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. See Figure 9 for illustrations of various overhang constructions.

The protection afforded by overhangs extends the life of the wall below, particularly if the wall is constructed of wood materials. As a reasonable guideline (given that in many cases no overhang is provided), protective overhang widths should be 12" to 24" in moist, humid climates and more if practicable. A reasonable practice is to provide a minimum of 12" 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).

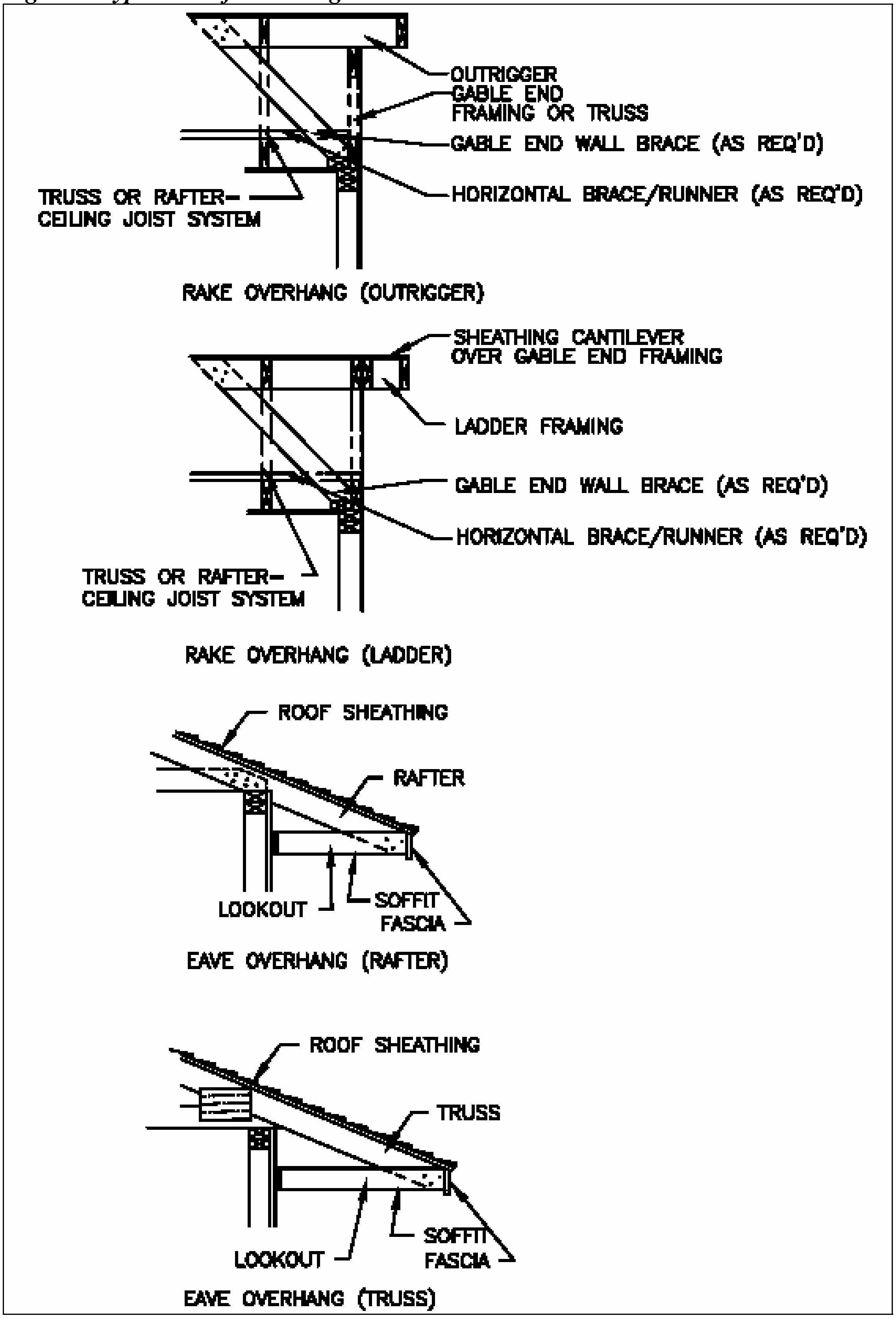
6.6 Gable-End Wall Bracing

Roof framing provides lateral support to the top of the walls where trusses and rafters are attached to the wall top plate. Floor framing provides lateral support to the top and bottom of walls, including the top of foundation walls. At a gable end 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. 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 9. Braces may be laid flat wise 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 engineering judgment should be exercised in this framing application with respect to the application of deflection criteria. The system deflection adjustment factors of Table 6 may assist in dealing with the need to meet a reasonable serviceability limit for deflection (see section 3.2).

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

Figure 9 Typical Roof Overhang Construction



Design Examples

Below are a number of design examples illustrate the design of various elements discussed in this course. The examples ended to provide practical advice and reference. The examples include notes and recommendations to improve the practicality and function of various possible design solutions. They are also intended to promote the engineer's creativity in arriving at the best possible solution for a particular application

EXAMPLE 1 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 2.4 and NDS 2.3)

C_D = 1.0 (Section 2.4.1)
C_r = 1.15 (Table 4)
C_F = 1.2 (NDS-S Table 4A adjustment factors)
C_H = 2.0 (Section 2.4.3)
C_L = 1.0 (NDS 3.3.3, continuous lateral support)
C_b = 1.0 (NDS 2.3.10)
F_b' = $F_bC_rC_FC_DC_L$ = 975 (1.15)(1.2)(1.0)(1.0) = 1,345 psi
F_v' = $F_vC_HC_D$ = 75 (2)(1.0) = 150 psi
$F_{c\perp}'$ = $F_{c\perp}C_b$ = 405 (1.0) = 405 psi
E' = E = 1,500,000 psi

3. Calculate the applied load

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

4. Determine maximum clear span based on bending capacity

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

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

$f_b \leq F_b'$
$6.08\ \ell^2 \leq 1{,}345$ psi
$\ell^2 = 221$
$\ell = 14.9$ ft = 14 ft-11 in (maximum clear span due to bending stress)

5. Determine maximum clear span based on horizontal shear capacity

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

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

$f_v \leq F_v'$

$3.7\ell \leq 150$ psi

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

6. Determine maximum clear span based on bearing capacity

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

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

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

$8.9\ell \leq 405$ psi

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

7. Consider maximum clear span based on deflection criteria (Section 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 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 3.2 recommends using a 40 psf design floor live load with the ℓ/360 deflection limit. Given that the span will not be greater than 15 feet, it is not necessary to use the absolute deflection limit of 0.5 inch.

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

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

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

*applied live load of 40 psf only

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

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

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

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

Conclusion

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

For instructional reasons, shrinkage across the depth of the floor joist or floor system may be estimated as follows based on the equations in Section 3.2:

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

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

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

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

EXAMPLE 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

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

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

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

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

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

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

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

6. Determine minimum modulus of elasticity due to vibration

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

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

Bending

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

Horizontal shear

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

Bearing

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

Minimum unadjusted tabulated properties required

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

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

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

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

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

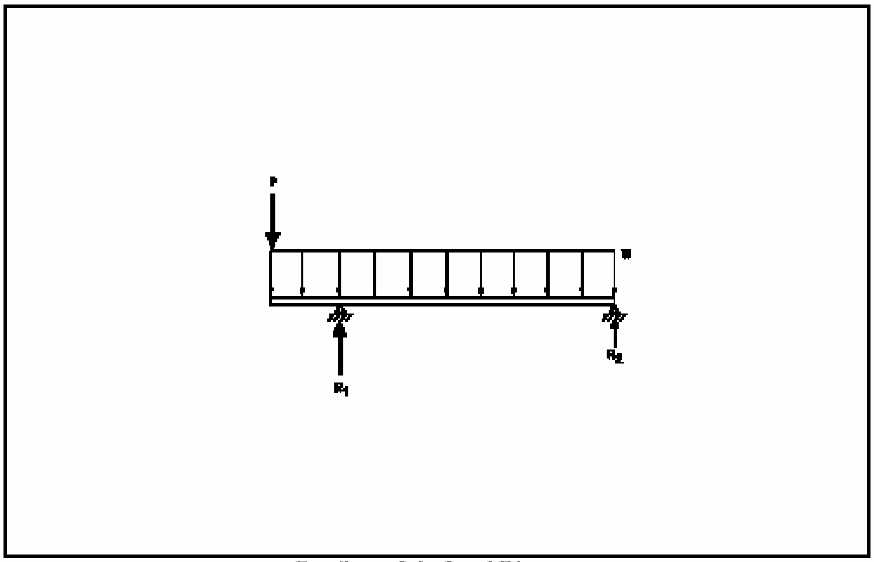
Conclusion

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

EXAMPLE 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
  Floor live load (L) = 40 psf
  Floor dead load (D) = 10 psf
Loads for concentrated load at end of cantilever
  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:

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 = $1.7x10^6$ 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 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).

$F_b' = F_bC_rC_FC_DC_L$ = (1000 psi)(1.15)(1.1)(1.25)(1.0) = 1,581 psi
$F_v' = F_vC_HC_D$ = (95)(2)(1.25) = 238 psi
$F_{c\perp}' = F_{c\perp}C_b$ = 625 (1.11) = 694 psi
$E' = E = 1.7x10^6$ psi

3. Determine design loads on cantilever joist

The following load combinations will be investigated
for several load cases that may govern different safety or serviceability checks

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

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

Case III: D+S+0.3L or D+L+0.3S - Bending and Horizontal Shear at Exterior Bearing Support
a. P = *f*(D+S)
w = *f*(D + 0.3L)

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

8. Check bearing stress

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

$$= 207 \text{ psi}$$

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

Conclusion

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

EXAMPLE 4 Built-Up Floor Girder Design

Given

Loads

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

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

Solution

1. Calculate the design load

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

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

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

3. Lumber property adjustments (Section 5.2.4):

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

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

F_b' = $F_bC_DC_rC_FC_L$ = 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 = 1.7×10^6 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 $S = 5(21.39) = 107 < 118$ (marginal, but 5 too thick)
4 2x12s $S = 4(31.64) = 127 > 118$ (OK)

5. Determine number of members required due to horizontal shear

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

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

$$f_v \leq F_v'$$

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

$A = 28\text{ in}^2$ 2 2x12s $A = 33.8 > 28$ OK
2 2x10s $A = 27.8 \approx 28$ OK

6. Determine required bearing length using 4 2x12s

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

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

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

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

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

7. Determine member size due to deflection

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

*includes 40 psf live load only

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

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

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

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

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

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

3 2x12s I = 534 > 519 okay

8. Check girder for floor system vibration control (see Section 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 $\ell/480$

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

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

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

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

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

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

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 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

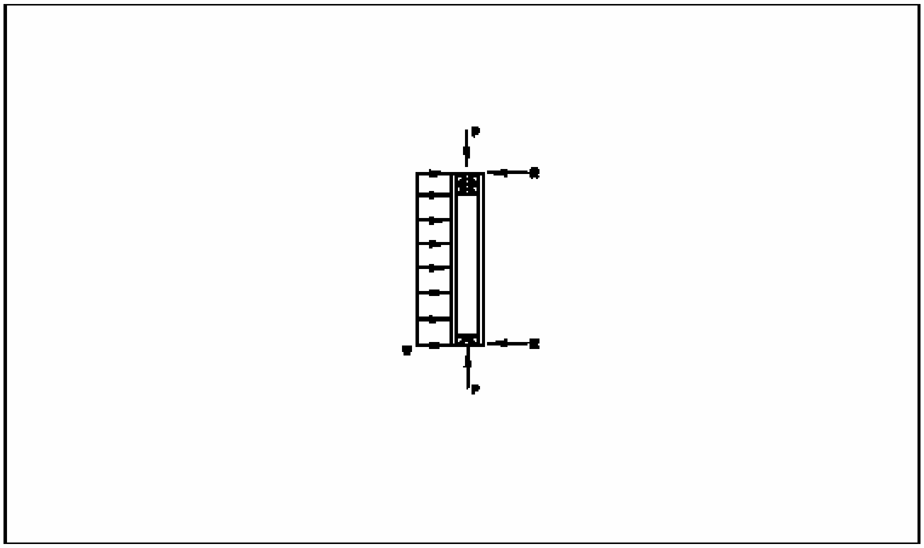
Sheathing design involves matching the proper sheathing rating with the floor framing spacing and live load condition. The process is generally a "cook book" method that follows tables in manufacturer's literature or the applicable building code. Board sheathing and decking are other possible subfloor 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 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

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), for applicable load combinations.



Wall Loading Diagram

Solution

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

F_b = 675 psi $F_{c\perp}$ = 425 psi
F_t = 350 psi F_c = 725 psi
F_v = 70 psi E = 1.2×10^6 psi

2. Determine lumber property adjustments (see Section 2.4)

C_D = 1.6 (wind load combination)
= 1.25 (gravity/snow load combination)
C_r = 1.5 (sheathed wall assembly, Table 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

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

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

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

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

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

$P = f_c A = (163 \text{ psi/stud})(1.5 \text{ in})(3.5 \text{ in}) = 856 \text{ lb/stud}$

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

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

7. Determine maximum axial gravity load without bending load

This analysis applies to the D + L+ 0.3(S or L_r) and D + (S or L_r) + 0.3L load combinations .

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.

$$F_c \le F_c'$$

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

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

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

8. Check bearing capacity of wall plate

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

Conclusion

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

Note:

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

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

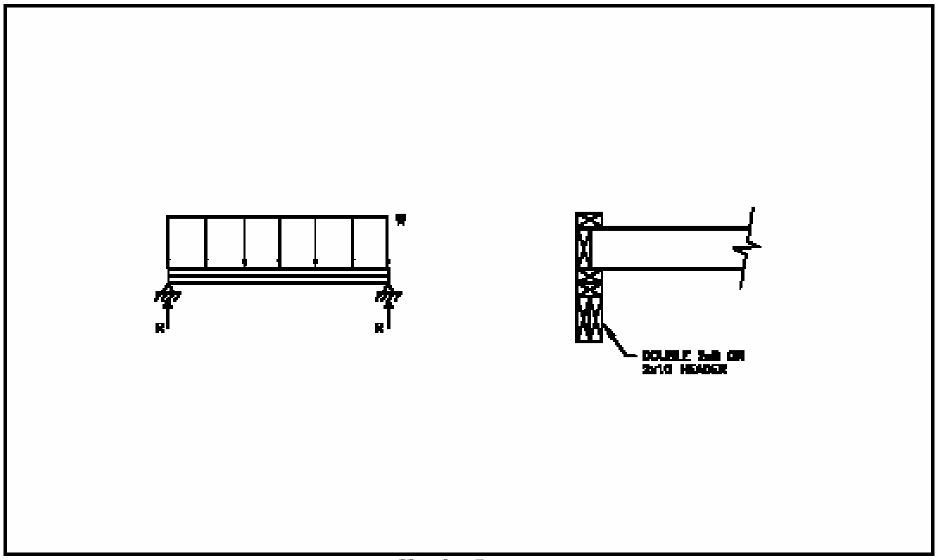
EXAMPLE 7 Header System Design

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)*

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 2.4)

C_r = 1.3 (2x10 double header per Table 8)
= 1.2 (2x8 double header per Table 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 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

$$M_{max} = \frac{w\ell^2}{8} = \frac{(600\ \text{plf})(6.5\ \text{ft})^2}{8} = 3{,}169\text{ ft-lb}$$

$$f_b = \frac{M_{max}}{S} \le F_b'$$

$$1{,}454\text{ psi} = \frac{3{,}169\text{ ft}-\text{lb}(12\ \text{in/ft})}{S}$$

$S = 26.2\text{ in}^3$
S for 2 2x10 $= 2(21.39\text{ in}^3) = 42.78\text{ in}^3 > 26.2\text{ in}^3$ (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 for 2 2x8 $= 2\,(13.14) = 26.3\text{ in}^3 < 28.3\text{ in}^3$ (close, but no good)

4. Determine member size due to bending for combined floor and supported wall loads by using the 1.8 system factor from Table 8, but not explicitly calculating the load sharing with the band joist above.

$F_b' = F_b(C_D)(C_r)(C_F)(C_L) = 775\text{ psi }(1.25)(1.8)(1.1)(1.0) = 1{,}918\text{ psi}$

$$M_{max} = \frac{w\ell^2}{8} = \frac{(360\ \text{plf} + 600\ \text{plf})(6.5\ \text{ft})^2}{8} = 5{,}070\text{ ft-lb}$$

$$f_b = M/S \le F_b'$$

$$1{,}918\text{ psi} = \frac{5{,}070\ \text{ft}-\text{lb}\ (12\ \text{in/ft})}{S}$$

$S = 31.7\text{ in}^3$
S for 2-2x10 $= 42.78\text{ in}^3 > 31.7\text{ in}^3$ (OK)

5. Check horizontal shear

$$V_{max} = \frac{w\ell}{2} = \frac{(600\ \text{plf})(6.5)}{2} = 1{,}950\text{ lb}$$

$$f_v = \frac{3V}{2A} = \frac{3(1{,}950\ \text{lb})}{2(2)(1.5\ \text{in})(9.25\ \text{in})} = 106\text{ psi}$$

$f_v \le F_v'$

106 psi < 175 psi (OK)

6. Check for adequate bearing

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

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

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

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

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

7. Check deflection

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

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

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

Conclusion

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

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

$$F_{cE} = \frac{K_{cE}E'}{\left(\ell_e/d\right)^2} = \frac{0.3(1.4\times10^6\text{psi})}{\left(7.3\text{ ft}(12\text{ in/ft})/3.5\text{ in}\right)^2} = 670\text{ psi}$$

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

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

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

Conclusion

A 4x4 column is adequate for the 4,800-pound axial design load and the stated height and support conditions. In fact, a greater column spacing could be used. Note that the analysis was performed with a solid sawn column of rectangular dimension. If a nonrectangular column is used, buckling must be analyzed in the weak-axis direction in consideration of the distance between lateral supports, if any, in that direction. If a built-up column is used, it is NOT treated the same way as a solid column. Even if the dimensions are nearly the same, the built-up column is more susceptible to buckling due to slippage between adjacent members as flexure occurs in response to buckling (only if unbraced in the weak-axis direction of the built-up members). Slippage depends on how well the built-up members are fastened together, which is accounted for by the use of an additional adjustment (reduction) factor applied to the C_p equation (see Section 5.5 and NDS 15.3).

EXAMPLE 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 7 for load diagram).

Solution

1. Evaluate load combinations applicable to rafter design:

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 x 10^6 psi

3. Determine relevant adjustments to property values assuming a 2x8 will be used (Section 2.4):

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

*Determined in accordance with NDS 3.3.3

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

4. Determine rafter transverse bending load, shear, and moment for the wind uplift load case (using Method A of Figure 8).

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

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

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

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

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

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

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

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

Outward Bending (0.6D + W_u)

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

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

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

Inward Bending (D + S)

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

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

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

7. Check horizontal shear

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

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

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

8. Check bearing

OK by inspection.

9. Check deflection criteria for gravity load condition (Section 2.2)

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

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

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

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

Conclusion

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

EXAMPLE 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.

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

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

$D + W$ Not controlling because snow load is greater in the inward direction; also, positive pressure is possible only on the sloped windward roof surface while the leeward roof surface is always under negative (suction) pressure for wind perpendicular to the ridge; the case of wind parallel to the ridge results in uplift across both sides of the roof, which is addressed in the $0.6\ D + W_u$ load combination and the roof uplift coefficients and based on 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 9, the following ridge beam loads are determined:

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

Load on ridge beam

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

0.6 w_{dead} = 121 plf

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

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

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

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

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

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

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

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

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

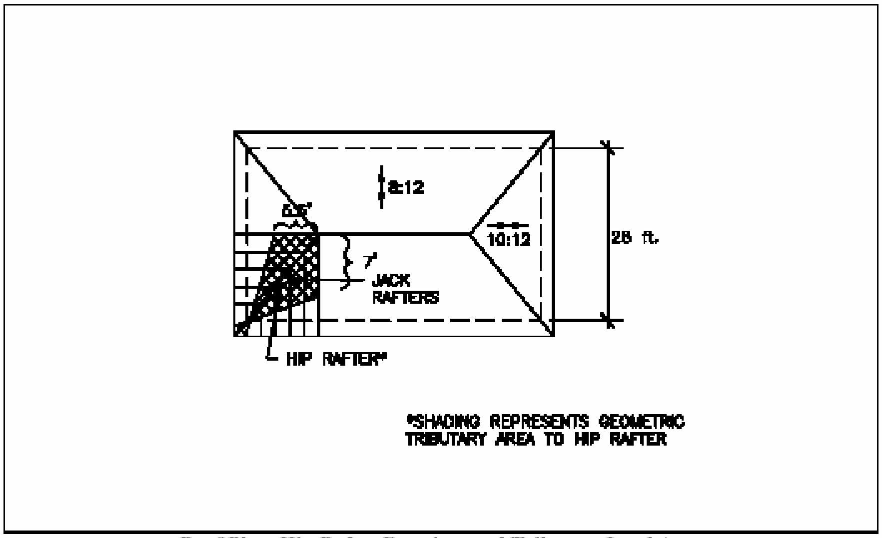
Note. The remainder of the problem is essentially identical to Example 9 with respect to determining the strength of the wood member. However, a trial member size and grade are needed to determine the lumber stresses as well as the lumber property adjustment values. Thus, the process of optimizing a 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 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.

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.

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

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

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

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

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

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

Tributary width ≈ 4 ft

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

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

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

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

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

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

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

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

$F_b' = F_bC_DC_rC_FC_L$ (F_b from NDS-S, Table 4A)

F_b' = 850 psi (1.25)(1.0)(1.0)(1.0) = 1,063 psi

$f_b \leq F_b'$

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

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

S_{2x12} = 31.6 in^3

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

Try 2-2x10s,

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

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

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

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

Therefore, 2-2x10s are acceptable ($2x21.39\text{ in}^3 = 42.8\text{ in}^3$).

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:

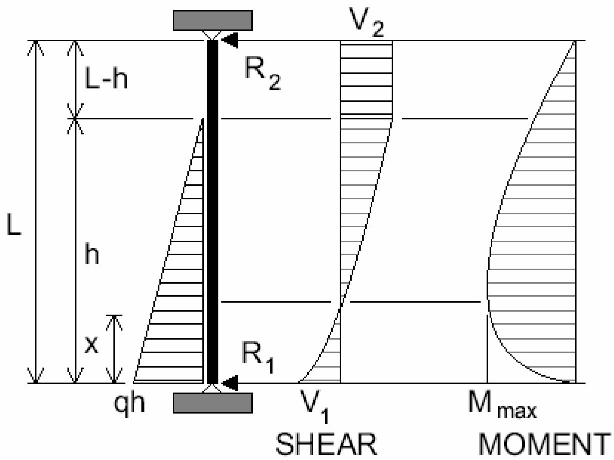
Deflection is OK by inspection. No method exists to accurately estimate deflection of a hip rafter that is subject to significant system stiffness because of the folded-plate action of the roof sheathing diaphragm.

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.

Appendix A

Shear and Moment Diagrams and Beam Equations



q = equivalent fluid density of soil (pcf)

qh = soil presure (psf) at x = 0

$$V_2 = -R_2 = \frac{-qh^3}{6L}$$

$$V_1 = R_1 = \frac{1}{2}qh^2\ (1-\frac{h}{3L})$$

$$V_x = V_1 - \frac{1}{2}xq\,(2h - x)\ \text{(where } x < h)$$

$$V_x = V_2\ \text{(where } x \geq h)$$

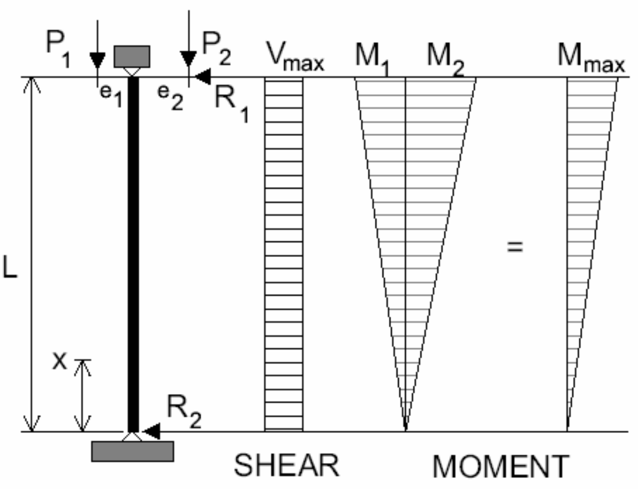
$$M_x = V_1 x - \frac{1}{2}qhx^2 + \frac{1}{6}qx^3\quad \text{(where } x < h)$$

$$M_x = -V_2\,(L - x)\ \text{(where } x \geq h)$$

$$x_{@M_{max}} = h - \sqrt{h^2 - \frac{2V_1}{q}}$$

$$\Delta_{max}\ (\text{at } x \cong L/2) \cong \frac{qL^3}{EI}\left[\frac{hL}{128} - \frac{L^2}{960} - \frac{h^2}{48} + \frac{h^3}{144L}\right]$$

Figure A.1 - Simple Beam (Foundation Wall) - Partial Triangular Load



$$V_{max} = R_2 = \frac{M_{max}}{L}$$

$$M_1 = P_1 e_1$$

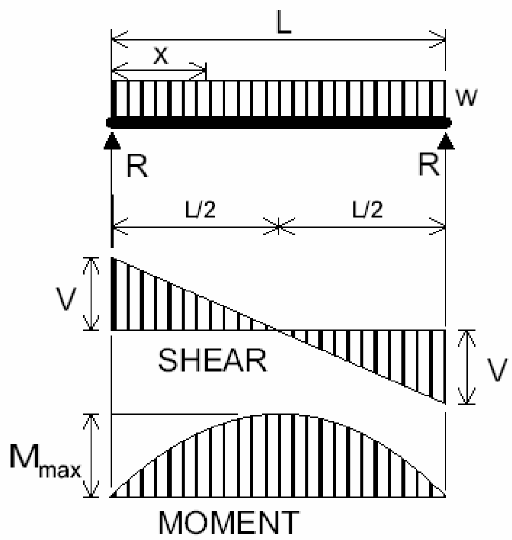
$$M_2 = P_2 e_2$$

$$M_{max} = |M_2| - |M_1| \text{ where } |M_2| > |M_1|$$

$$M_{max} = |M_1| - |M_2| \text{ where } |M_1| > |M_2|$$

$$M_x = M_{max}\left(\frac{x}{L}\right)$$

Figure A.2 - Simple Beam (Wall or Column) - Eccentric Point Loads



$$R = V_{max} = \frac{wL}{2}$$

$$V_x = w\left(\frac{L}{2} - x\right)$$

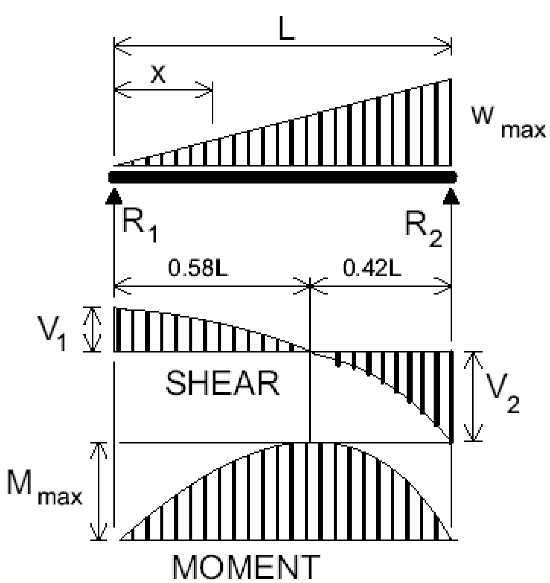
$$M_{max} = \frac{wL^2}{8} \text{ (at } x = \frac{L}{2})$$

$$M_x = \frac{wx}{2}(L - x)$$

$$\Delta_{max} = \frac{5wL^4}{384EI} \text{ (at } x = \frac{L}{2})$$

$$\Delta_x = \frac{wx}{24EI}\left(L^3 - 2Lx^2 + x^3\right)$$

Figure A.3 - Simple Beam - Uniformly Distributed Load



$$R_1 = V_1 = \frac{w_{max}}{3}$$

$$R_2 = V_2 = \frac{2w_{max}}{3}$$

$$V_x = \frac{w_{max}}{3} - \frac{w_{max}{}^2}{L^2}$$

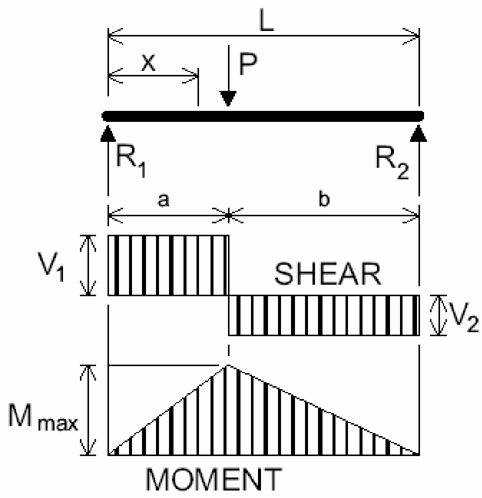
$$M_{max} \text{ (at } x = \frac{L}{\sqrt{3}}) = \frac{2w_{max}L}{9\sqrt{3}}$$

$$M_x = \frac{w_{max}x}{3L^2}(L^2 - x^2)$$

$$\Delta_{max} \text{ (at } x = L\sqrt{1 - \sqrt{\frac{8}{15}}}) = \frac{w_{max}L^3}{77EI}$$

$$\Delta_x = \frac{w_{max}x}{180EIL^2}(3x^4 - 10L^2x^2 + 7L^4)$$

Figure A.4 - Simple Beam - Load Increasing Uniformly to One End



$R_1 = V_1 \text{ (max when } a<b) = \dfrac{Pb}{L}$

$R_2 = V_2 \text{ (max when } a>b) = \dfrac{Pa}{L}$

$M_{max} \text{ (at point of load)} = \dfrac{Pab}{L}$

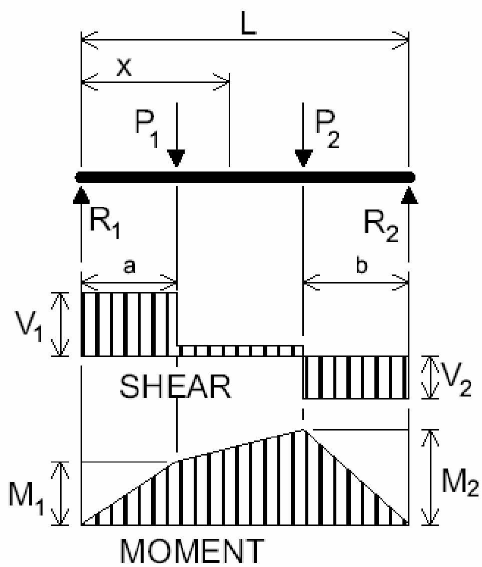
$M_x \text{ (when } x<a) = \dfrac{Pbx}{L}$

$\Delta_{max} \left[\text{at } x = \sqrt{\dfrac{a(a+2b)}{3}} \text{ when } a<b\right] = \dfrac{Fab(a+2b)\sqrt{3a(a+2b)}}{27EIL}$

$\Delta_a \text{ (at point of load)} = \dfrac{Pa^2b^2}{3EIL}$

$\Delta_x \text{ (when } x<a) = \dfrac{Pbx}{6EIL}(L^2 - b^2 - x^2)$

Figure A.5 - Simple Beam - Concentrated Load at Any Point



$R_1 = V_1 = \dfrac{P_1(L-a)+P_2 b}{L}$

$R_2 = V_2 = \dfrac{P_1 a + P_2(L-b)}{L}$

$V_x\ [\text{when } a<x<(L-b)] = R_1 - P_1$

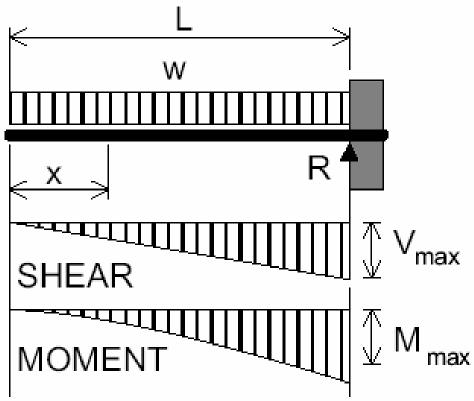
$M_1 \text{ (max when } R_1<P_1) = R_1 a$

$M_2 \text{ (max when } R_2<P_2) = R_2 b$

$M_x \text{ (when } x<a) = R_1 x$

$M_x\ [\text{when } a<x<(L-b)] = R_1 x - P_1(x-a)$

Figure A.6 - Simple Beam - Two Unequal Concentrated Loads Unsymmetrically Placed



$R = V_{max} = wL$

$V_x = wx$

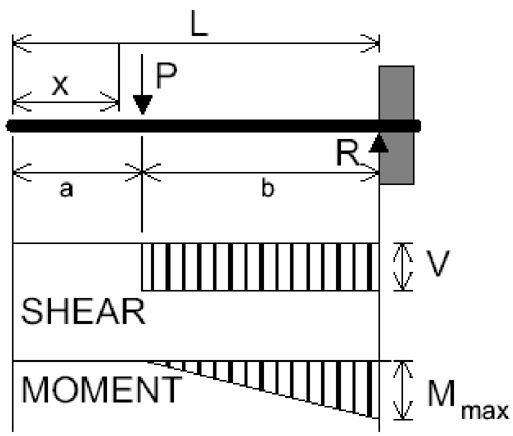
$M_{max} \text{ (at fixed end)} = \dfrac{wL^2}{2}$

$M_x = \dfrac{wx^2}{2}$

$\Delta_{max} \text{ (at free end)} = \dfrac{wL^4}{8EI}$

$\Delta_x = \dfrac{w}{24EI}(x^4 - 4L^3x + 3L^4)$

Figure A.7 - Cantilever Beam - Uniformly Distributed Load



$R = V = P$

$M_{max} \text{ (at fixed end)} = Pb$

$M_x \text{ (when x>a)} = P(x\text{-}a)$

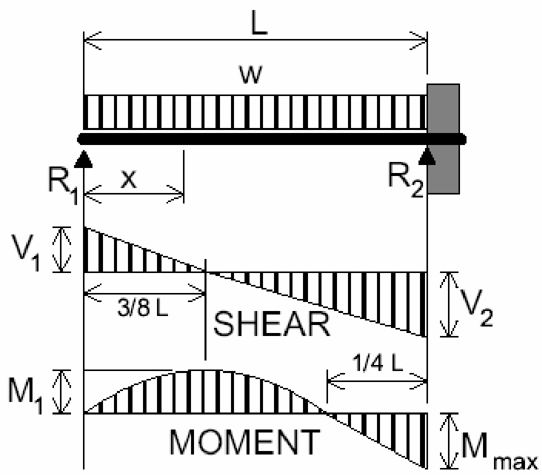
$\Delta_{max} \text{ (at free end)} = \dfrac{Pb^2}{6EI}(3L\text{-}b)$

$\Delta_a \text{ (at point of load)} = \dfrac{Pb^3}{3EI}$

$\Delta_x \text{ (when x<a)} = \dfrac{Pb^2}{6EI}(3L\text{-}3x\text{-}b)$

$\Delta_x \text{ (when x>a)} = \dfrac{P(L-x)^2}{6EI}(3b\text{-}L\text{+}x)$

Figure A.8 - Cantilever Beam - Concentrated Load at Any Point



$$R_1 = V_1 = \frac{3wL}{8}$$

$$R_2 = V_2 = V_{max} = \frac{5wL}{8}$$

$$V_x = R_1 - wx$$

$$M_{max} = \frac{wL^2}{8}$$

$$M_1 \text{ (at } x = \text{)} = \frac{3}{8}L = \frac{9}{128}wL^2$$

$$M_x = R_1 x - \frac{wx^2}{2}$$

$$\Delta_{max} \text{ (at } x = \frac{L}{16}(1 + \sqrt{33}) = 0.42L) = \frac{wL^4}{185EI}$$

$$\Delta_x = \frac{wx}{48EI}(L^3 - 3Lx^2 + 2x^3)$$

Figure A.9 - Beam Fixed at One End, Supported at Other - Uniformly Distributed Load

TABLE 1.1 Typical Load Combinations Used for the Design of Components and Systems

Component or System	ASD Load Combinations	LRFD Load Combinations
Foundation wall (gravity and soil lateral loads)	D + H D + H + L + 0.3 (L_r + S) D + H + (L_r or S) + 0.3 L	1.2D + 1.6H 1.2D + 1.6H + 1.6L + 0.5(L_r + S) 1.2D + 1.6H + 1.6(l_r or S) + 0.5L
Headers, girders, joists, interior load-bearing walls and columns, footings (gravity loads)	D + L + 0.3 (L_r + S) D + (L_r or S) + 0.3 L	1.2D + 1.6L^2 + 0.5(L_r + S) 1.2D + 1.6(l_r or S) + 0.5L
Exterior load-bearing walls and columns (gravity and transverse lateral load)	Same as immediately above plus D + W D + 0.7E + 0.5L + 0.2S	Same as immediately above plus 1.2D + 1.5W 1.2D + 1.0E + 0.5L + 0.25
Roof rafters, trusses, and beams: roof and wall sheathing (gravity and wind loads)	D + (L_r or S) 0.6D + W_u D + W	1.2D + 1.6(L_r or S) 0.9D + 1.5W_u 1.2D + 1.5W
Floor diaphragms and shear walls (in-place lateral and overturning loads)	0.6D + (W or 0.7E)	0.9D + (1.5W or 1.0E)

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

1.3 Dead Loads

Dead loads are made up of the permanent construction material loads composing the roof, floor, wall, and foundation systems, including claddings, finishes, and fixed equipment. The values for dead loads in Table 1.2 are for commonly used materials and construction in light-frame residential buildings. Table 1.3 provides values for common material densities and may be useful to calculate dead loads more accurately. The design examples in this course will demonstrate a straightforward process of calculating dead loads.

TABLE 1.2 Dead Loads for Common Residential Construction

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

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

TABLE 1.3 Densities for Common Residential Construction Materials

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

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

1.4 Live Loads

Live loads are created by the use and occupancy of a building. Loads include human occupants, furnishings, moveable equipment, storage, and construction and maintenance activities. Table 1.4 provides recommended design live loads for residential buildings. Example 1.1 will demonstrate the use of those loads and the load combinations specified in Table 1.1, along with other factors discussed here. To adequately define the loading condition, loads are presented in terms of uniform area loads (psf), concentrated loads (lbs), and uniform line loads (plf). The uniform and concentrated live loads should not be applied simultaneously in a structural evaluation. Concentrated loads should be applied to a small area or surface consistent with the application and should be located or directed to give the maximum load effect possible in end-use conditions. For example, the stair concentrated load of 300 pounds should be applied to the center of the stair tread between supports. The concentrated wheel load of a vehicle on a garage slab or floor should be applied to all areas or members subject to a wheel load, using a loaded area of about 20 square inches.

TABLE 1.4 Live Loads for Residential Construction

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

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

The floor live load on any given floor area may be reduced in accordance with Equation 14.1. The equation applies to floor and support members, such as beams or columns that experience floor loads from a total tributary floor area greater than 200 square feet. This equation is different from what is found in most engineering manual since it is based on data that applies to residential floor loads rather than commercial buildings.

Equation 1.4.1

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

where,

L = the adjustment floor live load for tributary areas greater than 200 square feet
A_t = the tributary from a single-story area assigned to a floor support member (i.e., girder, column, or footing)
L_o = the unreduced live load associated with a floor area of 200 ft^2 from Table 1.4

The nominal design floor live load in Table 1.4 includes both a sustained and transient load component. The sustained component is that load typically present at any given time and includes the load associated with normal human occupancy and furnishings. For residential buildings, the mean sustained live load is about 6 psf and can very from 4 to 8 psf. The mean transient live load for dwellings is also about 6 psf but could very to a high of 13 psf. A total design live load of 30 to 40 psf is conservative.

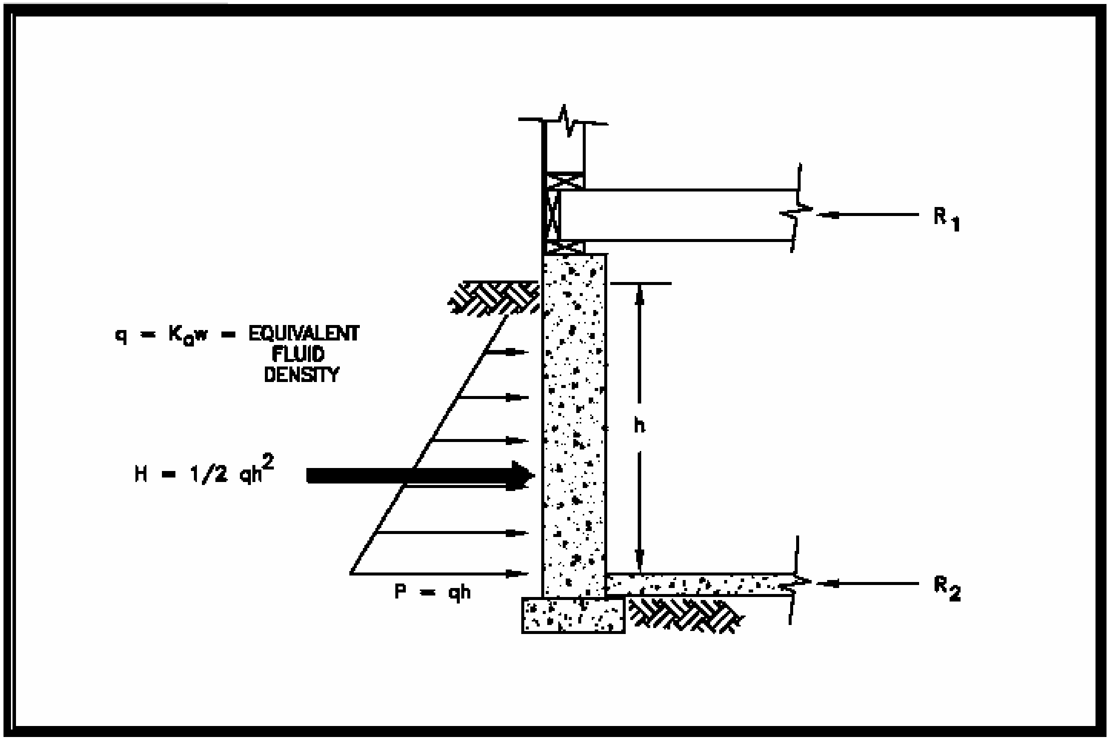
1.5 Soil Lateral Loads

Lateral pressure exerted by the earth backfill against a residential foundation wall (basement wall) can be calculated with reasonable accuracy on the basis of theory and for conditions that rarely occur in practice. Theoretical analyses are usually based on homogeneous materials that demonstrate consistent compaction and behavioral properties. These conditions are rarely experienced in residential construction projects. A more common method of determining lateral soil loads on residential foundations is the Rankine's (1857) theory of earth pressure and uses the Equivalent Fluid Density (EFD) method shown in Figure 1.1, where pressure distribution is assumed to be triangular and increases with depth. In the EFD method, the soil unit weight W is multiplied by an empirical coefficient Ka to account for soil not actually being fluid and that the pressure distribution is not necessarily triangular. The coefficient Ka is known as the active Rankine pressure coefficient. Thus, the equivalent fluid density (EFD) is determined as follows:

Equation 1.5.1

$$q = K_a w$$

Figure 1.1, Triangular Pressure Distribution on a Basement Foundation Wall



For the triangular pressure distribution shown in Figure 1.1, the pressure at depth, h, in feet is

Equation 1.5.2

$$P = qh$$

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

Equation 1.5.3

$$H = \frac{1}{2}(qh)(h) = \frac{1}{2}qh^2$$

where,

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

The EFD method is subject to judgment as to the appropriate value of the coefficient Ka. The values of Ka in Table 1.5 are recommended for the determination of lateral pressures on residential foundations for various types of backfill materials placed with light compaction and good drainage. Given the long time use of a 30 pcf equivalent fluid density in residential foundation wall prescriptive design tables, the values in Table 1.5 are considered somewhat conservative for typical conditions. A relatively conservative safety factor of 3 to 4 is typically applied to the design of unreinforced or nominally reinforced masonry or concrete foundation walls. Therefore, at imminent failure of a foundation wall, the 30 psf design EFD would correspond to an active soil lateral pressure determined by using an equivalent fluid density of about 90 to 120 pcf or more.

TABLE 1.5
Values of Ka , Soil Unit Weight, and Equivalent Fluid Density by Soil Type

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

Notes:
Values are applicable to well-drained foundations with less than 10 feet of backfill placed with light compaction or natural settlement as is common in residential construction. The values do not apply to foundation walls in flood-prone environments. In such cases, an equivalent fluid density value of 80 to 90 pcf would be more appropriate.
These values do not consider the significantly higher loads that can result from expansive clays and the lateral expansion of moist, frozen soil. Such conditions should be avoided by eliminating expansive clays adjacent to the foundation wall and providing for adequate surface and foundation drainage.
Organic silts and clays and expansive clays are unsuitable for backfill material.
Backfill in the form of clay soils (nonexpansive) should be used with caution on foundation walls with unbalanced fill heights greater than 3 to 4 feet and on cantilevered foundation walls with unbalanced fill heights greater than 2 to 3 feet.

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

1.6 Wind Loads

1.6.1 General

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

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

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

The next section presents a simplified method for determining both MWFRS and C&C wind loads. Since the loads in Section 1.6.2 are determined for specific applications, the calculation of MWFRS and C&C wind loads is implicit in the values provided. Design Example 1.2 demonstrates the calculation of wind loads by applying the simplified method of the following Section 1.6.2 to several design conditions associated with wind loads and the load combinations presented in Table 1.1.

1.6.2 Determination of Wind Loads on Residential Structures

The method for the design of residential buildings in this course is based on a simplification of the ASCE 7-98 wind provisions (ASCE, 1999); however, wind loads listed in ASCE 7-89 are not exact duplicate. Lateral loads and roof uplift loads are determined by using a projected area approach. Other wind loads are determined for specific components or assemblies that comprise the exterior building envelope. Five steps are required to determine design wind loads on a residential building and its components.

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

From the wind map in Figure 1.2 (refer to ASCE 7 for maps with greater detail), select a design wind speed for the site. The wind speeds may appear higher than those used in older design wind maps. The difference is due solely to the use of the "peak gust" to define wind speeds rather than an averaged wind speed as represented by the "fastest mile of wind" used in older wind maps. Nominal design peak gust wind speeds are typically 85 to 90 mph in most of the United States; however, along the hurricane-prone Gulf and Atlantic coasts, nominal design wind speeds range from 100 to 150 mph for the peak gust.

If relying on either an older fastest-mile wind speed map or older design provisions based on fastest-mile wind speeds, the engineer should convert wind speed in accordance with Table 1.6 for use with this simplified method, which is based on peak gust wind speeds.

TABLE 1.6 Wind Speed Conversions

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

Once the nominal design wind speed in terms of peak gust is determined, the engineer can select the basic velocity pressure in accordance with Table 1.7. The basic velocity pressure is a reference wind pressure to which pressure coefficients are applied to determine surface pressures on a building. Velocity pressures in Table 1.7 are based on typical conditions for residential construction, namely, suburban terrain exposure and relatively flat or rolling terrain without topographic wind speed-up effects.

FIGURE 1.2 Basic Design Wind Speed Map from ASCE 7-98

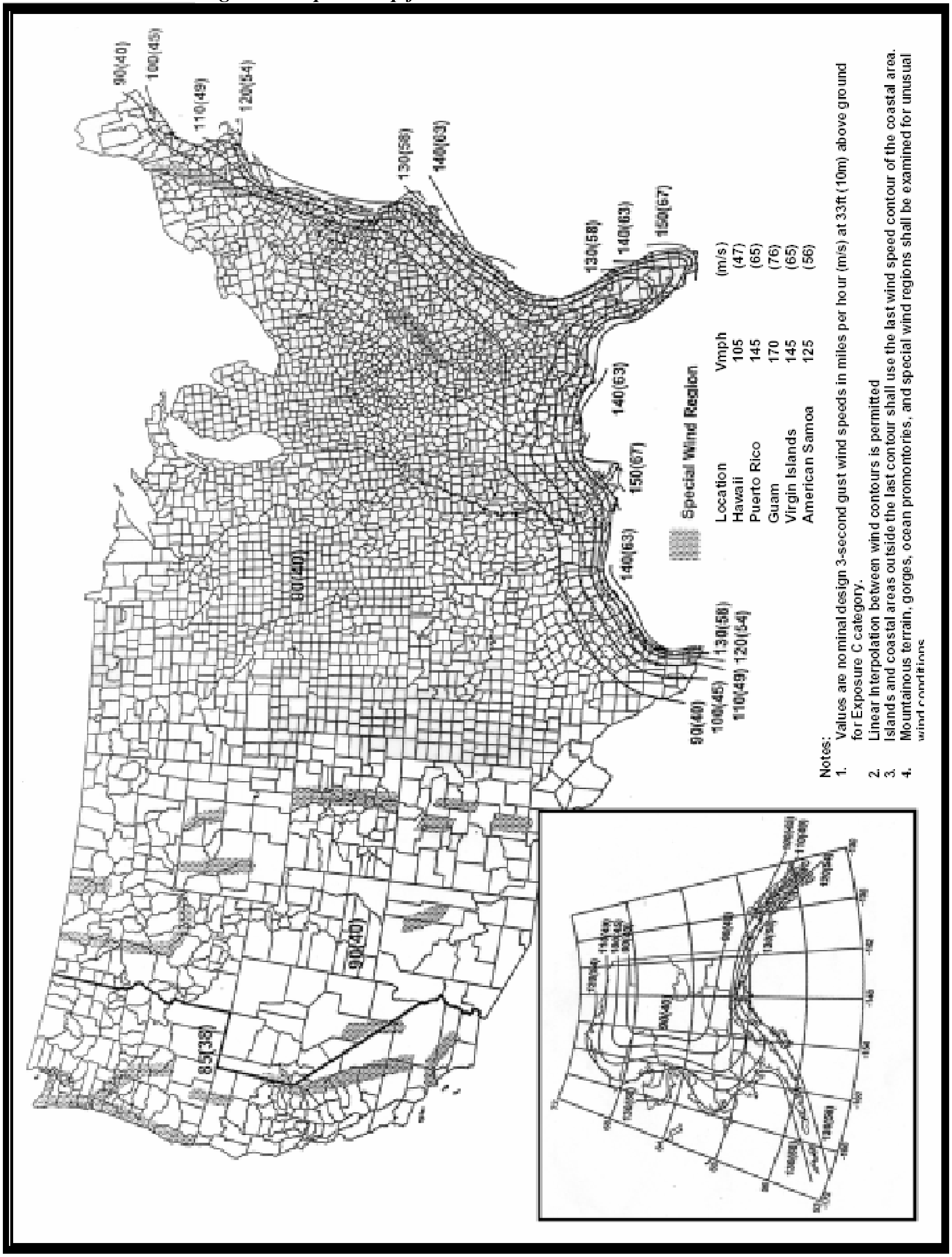


TABLE 1.7 Basic Wind Velocity Pressures (psf) for Suburban Terrain

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

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

Step 2: Adjustments to the basic velocity pressure

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

Open exposure. The wind values in Table 1.7 are based on typical residential exposures to the wind. If a site is located in generally open, flat terrain with few obstructions to the wind in most directions or is exposed to a large body of water (i.e., ocean or lake), the designer should multiply the values in Table 1.7 by a factor of 1.4. The factor may be adjusted for sites that are considered intermediate to open suburban exposures. It may also be used to adjust wind loads according to the exposure related to the specific directions of wind approach to the building. The wind exposure conditions used in this guide are derived from ASCE 7 with some modification applicable to small residential buildings of three stories or less.

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

Protected exposure. If a site is generally surrounded by forest or densely wooded terrain with no open areas greater than a few hundred feet, smaller buildings such as homes experience significant wind load reductions from the typical suburban exposure condition assumed in Table 1.7. If such conditions exist and the site's design wind speed does not exceed about 120 mph peak gust, the engineer may consider multiplying the values in Table 1.7 by 0.8. The factor may be used to adjust wind loads according to the exposure related to the specific directions of wind approach to the building. Wind load reductions associated with a protected exposure in a suburban or otherwise open exposure have been shown to approximate 20 percent. In densely treed terrain with the height of the building below that of the treetops, the reduction factor applied to Table 1.7 values can approach 0.6. The effect is known as shielding; however, ASCE 7 does not currently permit it. Two considerations require judgment: Are the sources of shielding likely to exist for the expected life of the structure? Are the sources of shielding able to withstand wind speeds in excess of a design event?

Wind directionality. As noted, the direction of the wind in a given event does not create peak loads (which provide the basis for design pressure coefficients) simultaneously on all building surfaces. In some cases, the pressure zones with the highest design pressures are extremely sensitive to wind direction. In accordance with ASCE 7, the velocity pressures in Table 1.7 are based on a directionality adjustment of 0.85 that applies to hurricane wind conditions where winds in a given event are multidirectional but with varying magnitude. However, in “straight” wind climates, a directionality factor of 0.75 has been shown to be appropriate. Therefore, if a site is in a nonhurricane-prone wind area (i.e., design wind speed of 110 mph gust or less), the engineer may also consider multiplying the values in Table 1.7 by 0.9 (i.e., 0.9 x 0.85 ≅ 0.75) to adjust for directionality effects in non-hurricane-prone wind environments.

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

Step 3: Determine lateral wind pressure coefficients

Lateral pressure coefficients in Table 1.8 are composite pressure coefficients that combine the effect of positive pressures on the windward face of the building and negative (suction) pressures on the leeward faces of the building. When multiplied by the velocity pressure from Steps 1 and 2, the selected pressure coefficient provides a single wind pressure that is applied to the vertical projected area of the roof and wall as indicated in Table 1.8. The resulting load is then used to design the home’s lateral force-resisting system. The lateral wind load must be determined for the two orthogonal directions on the building (i.e., parallel to the ridge and perpendicular to the ridge), using the vertical projected area of the building for each direction. Lateral loads are then assigned to various systems (e.g., shear walls, floor diaphragms, and roof diaphragms) by use of tributary areas.

TABLE 1.8 Lateral Pressure Coefficients for Application to Vertical Projected Areas

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

Step 4: Determine wind pressure coefficients for components and assemblies

The pressure coefficients in Table 1.9 are based on the assumption that the building is enclosed and not subject to higher internal pressures that may result from a windward opening in the building. The use of the values in Table 1.9 greatly simplifies a more detailed methodology described in most engineering manuals; as a result, there is some “rounding” of numbers. With the exception of the roof uplift coefficient, all pressures calculated with the coefficients are intended to be applied to the perpendicular building surface area that is tributary to the element of concern. Thus, the wind load is applied perpendicular to the actual building surface, not to a projected area. The roof uplift pressure coefficient is used to determine a single wind pressure that may be applied to a horizontal projected area of the roof to determine roof tie-down connection forces.

For buildings in hurricane-prone regions subject to wind-borne debris, the GCp values in Table 1.9 must be increased in magnitude by ±0.35 to account for higher potential internal pressures due to the possibility of a windward wall opening (i.e., broken window).

Step 5: Determine design wind pressures

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

TABLE 1-9 Wind Pressure Coefficients for Systems and Components (enclosed building)

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

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

1.6.3 Special Considerations in Hurricane-Prone Environments

1.6.3.1 Wind-Borne Debris

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

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

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

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

Current standards for selecting impact criteria for wind-borne debris protection do not explicitly consider all of the above factors. Further, the primary debris source in typical residential developments is asphalt roof shingles, which are not represented in existing impact test methods. These factors can have a dramatic effect on the level of wind-borne debris risk; moreover, existing impact test criteria appear to take a worst-case approach. Table 1.10 presents an example of missile types used for current impact tests. Additional factors to consider include emergency egress or access in the event of fire when impact-resistant glazing or fixed shutter systems are specified, potential injury or misapplication during installation of temporary methods of window protection, and durability of protective devices and connection details (including installation quality) such that they themselves do not become a debris hazard over time.

TABLE 1.10 Missile Types for Wind-Borne Debris Impact Tests

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

Notes:

These missile types are not necessarily representative of the predominant types or sources of debris at any particular site. Steel balls are intended to represent small gravels that would be commonly used for roof ballast. The 2x4 missiles are intended to represent a direct, end-on blow from construction debris without consideration of the probability of such an impact over the life of a particular structure.

Wind-borne debris regions are areas within hurricane-prone regions that are located (1) within one mile of the coastal mean high water line where the basic wind speed is equal to or greater than 110 mph or in Hawaii or (2) where the basic wind speed is equal to or greater than 120 mph. As outlined in Section 1.6.2 higher internal pressures are to be considered for buildings in wind-borne debris regions unless glazed openings are protected by impact-resistant glazing or protective devices proven as such by an approved test method. Approved test methods include ASTM E1886 and SSTD 12-97 (ASTM, 1997; SBCCI, 1997).

The wind load method described in Section 1.6.2 may be considered acceptable without wind-borne debris protection, provided that the building envelope (i.e., windows, doors, sheathing, and especially garage doors) is carefully designed for the required pressures. Most homes that experience windborne debris damage do not appear to exhibit more catastrophic failures, such as a roof blow-off, unless the roof was severely under designed in the first place (i.e., inadequate tie-down) or subject to poor workmanship (i.e., missing fasteners at critical locations). Those cases are often the ones cited as evidence of internal pressure in anecdotal field studies. Garage doors that fail due to wind pressure more frequently precipitate additional damage related to internal pressure. Because of these internal pressures, in hurricane regions, garage door reinforcement or pressure rated garage doors should be specified and their attachment to structural framing carefully considered.

1.6.3.2 Building Durability

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

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

1.6.3.3 Tips to Improve Performance

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

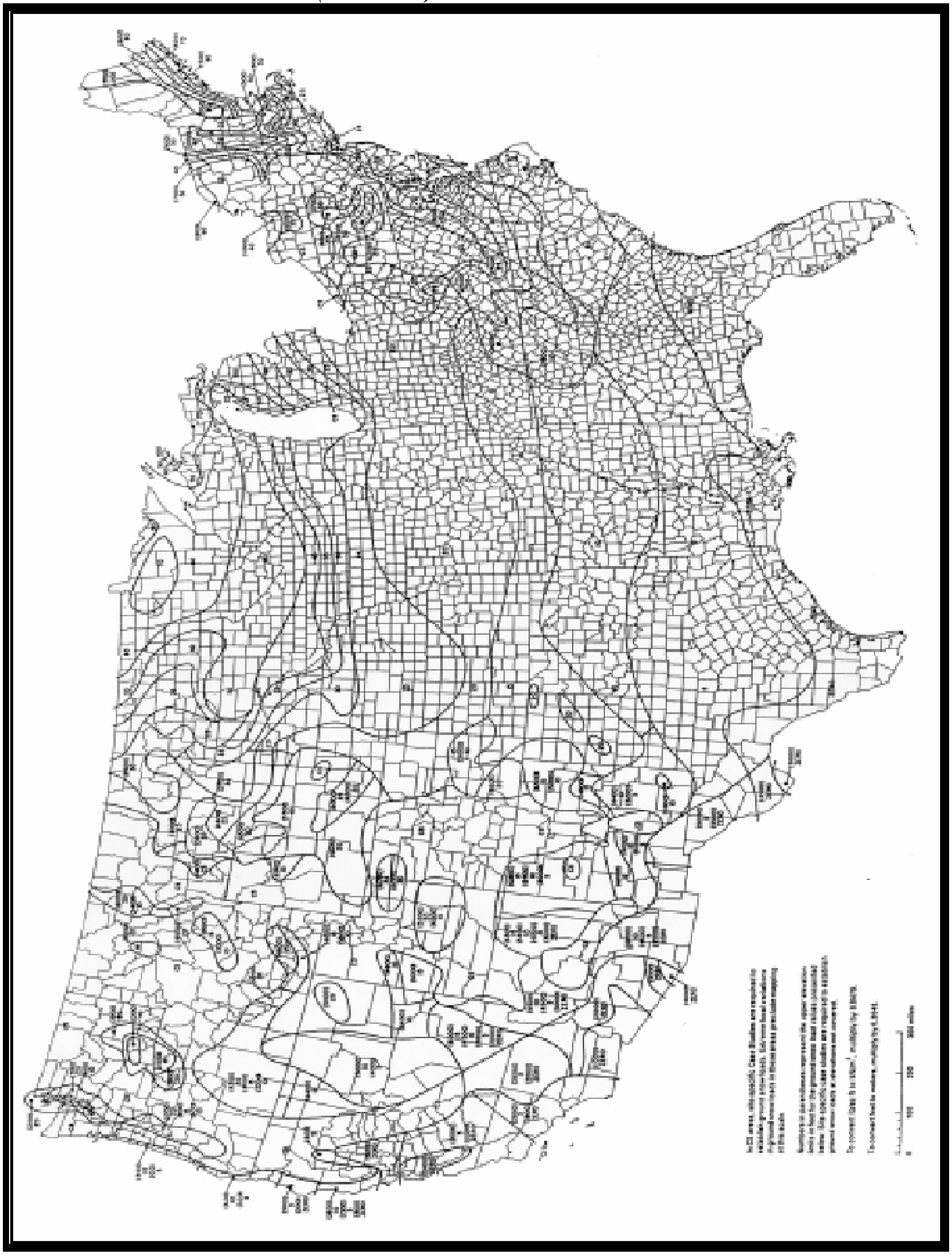
- One-story buildings are much less vulnerable to wind damage than two- or three-story buildings.
- On average, hip roofs have demonstrated better performance than gable-end roofs.
- Moderate roof slopes (i.e., 4:12 to 6:12) tend to optimize the trade-off between lateral loads and roof uplift loads (i.e., more aerodynamically efficient).
- Roof sheathing installation should be inspected for the proper type and spacing of fasteners, particularly at connections to gable-end framing.
- The installation of metal strapping or other tie-down hardware should be inspected as required to ensure the transfer of uplift loads.
- If composition roof shingles are used, high-wind fastening requirements should be followed (i.e., 6 nails per shingle in lieu of the standard 4 nails). A similar concern exists for tile roofing, metal roofing, and other roofing materials.
- Consider some practical means of glazed opening protection in the most severe hurricane-prone areas.

1.7 Snow Loads

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

Design ground snow loads may be obtained from the map in Figure 1.3; however, snow loads are most likely defined by the local building department. Typical ground snow loads range from 0 psf in the South to 50 psf in the northern United States. In mountainous areas, the ground snow load can surpass 100 psf. Local snow data should be carefully considered by the engineer. In areas where the ground snow load is less than 15 psf, the minimum roof live load (see to section 1.4) is usually the controlling gravity load in roof design. For a larger map with greater detail, refer to ASCE 7.

FIGURE 1.3 Ground Snow Loads (ASCE 7-98)



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1.8 Earthquake Loads

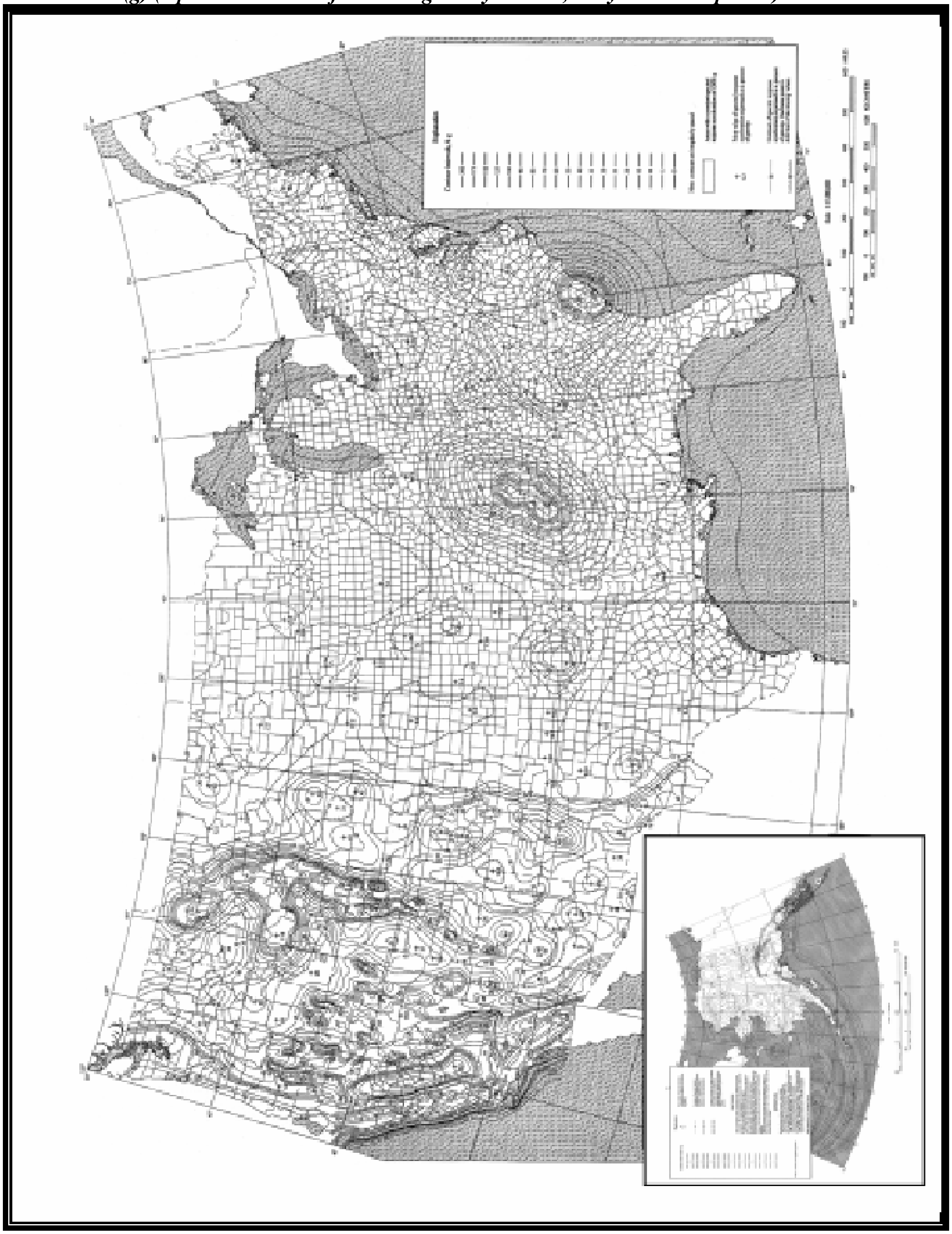
1.8.1 General

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

Most residential designs use a simplified approach similar to that in older seismic design codes. The approach outlined in the next section follows the older approach in terms of its simplicity while using the newer seismic risk maps and design format of NEHRP as incorporated into recent building code development efforts (ICC); see Figure 1.4. It should be noted, however, that the newer maps are not without controversy relative to seismic risk predictions, particularly in the eastern United States. For example, the maps are considered to overstate significantly the risk of earthquakes in the New Madrid seismic region around St. Louis, MO. Based on research and the manner of deriving the NEHRP maps for the New Madrid seismic region, the design seismic loads may be conservative by a factor of 2 or more. The engineer should bear in mind these uncertainties in the design process.

Wood-framed residential structures have performed well in major seismic events due to there their light-weight and resilient construction, the strength provided by nonstructural systems such as interior walls, and their load distribution capabilities. Only in the case of gross absence of good engineering judgment or misapplication of design for earthquake forces have severe life-safety consequences become an issue in light-frame, low-rise structures experiencing extreme seismic events.

FIGURE 1.4 Seismic Map of Design Short-Period Spectral Response Acceleration (g) (2 percent chance of exceeding a 50 year or 2,475-year return period)



1.8.2 Determination of Earthquake Loads on Houses

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

Equation 1.8.1

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

where,

S_{DS} = the design spectral response acceleration in the short-period range determined by Equation 1.8.2 (g)
R = the response modification factor (dimensionless)
W = the total weight of the building or supported by the story under consideration (lb); 20 percent of the roof snow load is also included where the ground snow load exceeds 30 psf
1.2 = factor to increase the seismic shear load based on the belief that the simplified method may result in greater uncertainty in the estimated seismic load

In calculating story shear for a given story, the engineer will apply to that story one-half of the dead load of the walls on the story under consideration and the dead load supported by the story. Dead loads used in determining seismic story shear or base shear are found in section 1.3. For housing, the interior partition wall dead load is effectively accounted for by the use of a 6 psf load distributed uniformly over the floor area. When applicable, the snow load may be determined in accordance with section 1.7. The inclusion of any snow load is based on the assumption that the snow is always frozen and adhered to the building such that it is part of the building mass during the entire seismic event.

The design spectral response acceleration for short-period ground motion SDS is used since light-frame buildings such as houses have a short period of vibration in response to seismic ground motion (i.e., high natural frequency). Nondestructive tests of existing houses have confirmed the short period of vibration, although once ductile damage has begun to occur in a severe event, the natural period of the building will increase. There are no valid methods available to determine the natural period of vibration for use in the seismic design of light-frame houses. Therefore, the short-period ground motion is used in the interest of following traditional practice.

Values of Ss are obtained from Figure 1.7. The value of SDS should be determined in consideration of the mapped short-period spectral response acceleration Ss and the required soil site amplification factor Fa as follows:

Equation 1.8.2

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

The value of Ss ranges from practically zero in low-risk areas to 3g in the highest-risk regions of the United States. A typical value in high seismic areas is 1.5g. It is to be noted that, wind loads control the design of the lateral force-resisting system of light-frame houses when Ss is less than about 1g. The 2/3 coefficient in Equation 1.8.2 is used to adjust to a design seismic ground motion value from that represented by the mapped Ss values (i.e., the mapped values are based on a "maximum considered earthquake" generally representative of a 2,475-year return period, with the design basis intended to represent a 475-year return period event).

Table 1.11 provides the values of Fa for a standard "firm" soil condition used for the design of residential buildings. Fa will decrease with increasing ground motion because the soil begins to dampen the ground motion as shaking intensifies. Because of this, the soil can have a moderating effect on the seismic shear loads experienced by buildings in high seismic risk regions. Dampening will also occur between a building foundation and the soil and will have a moderating effect. However, the soil-structure interaction effects on residential buildings have had little study; therefore, precise design procedures have not been developed. If a site is located on fill soils or "soft" ground, a different value of Fa should be considered. It has been learned, through experience, that soft soils do not affect the performance of the above ground house structure as much as they affect the site and foundations (e.g., settlement, fissuring, liquefaction, etc.).

TABLE 1.11 Site Soil Amplification Factor Relative to Acceleration (short period, firm soil)

S_s	$\leq$ 0.25g	0.5g	0.75g	1.0g	$\geq$ 1.25g
F_a	1.6	1.4	1.2	1.1	1.0

The seismic response modifier R has a long history in seismic design, but with little in the way of scientific underpinnings. In recognition that buildings can effectively dissipate energy from seismic ground motions through ductile damage, the R factor was developed to adjust the shear forces from that which would be experienced if a building could exhibit perfectly elastic behavior without some form of ductile energy dissipation. This has served a major role in standardizing the seismic design of buildings even though it has come about in the absence of a repeatable and generalized evaluation methodology with a known relationship to actual building performance.

Those structural building systems that are able to withstand greater ductile damage and deformation without substantial loss of strength are assigned a higher value for R. The R factor also incorporates differences in dampening that occur for various structural systems. Table 1.12 provides some values for R that should be used in residential design.

TABLE 1.12 Seismic Response Modifiers for Residential Construction

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

Notes:
The R-factors may vary for a given structural system type depending on wall configuration, material selection, and connection detailing, but these considerations are necessarily matters of designer judgment.
The R for light-frame shear walls (steel-framed and wood-framed) with shear panels has been recently revised to 6. Current practice typically uses an R of 5.5 to 6.5 depending on the edition of the local building code.
The wall is reinforced in accordance with concrete design requirements in ACI-318 or ACI-530. Nominally reinforced concrete or masonry that has conventional amounts of vertical reinforcement such as one #5 rebar at openings and at 4 feet on center may use the value for reinforced walls provided the construction is no more than two stories above grade.

Design Example 1.3 demonstrates the calculation of design seismic shear load based on the simplified procedures.

1.8.3 Seismic Shear Force Distribution

The vertical distribution of seismic forces to separate stories on a light-frame building is assumed to be in accordance with the mass supported by each story. Design codes vary in the requirements related to vertical distribution of seismic shear. There is no clear body of evidence to confirm any particular method of vertical seismic force distribution for light-frame buildings. So the engineer must keep with the simplified method given in Section 1.8.2, the approach used in this course reflects what is considered conventional practice. The horizontal distribution of seismic forces to various shear walls on a given story also varies in current practice for light-frame buildings. Several existing approaches to the design of the lateral force-resisting system of light-frame houses address the issue of horizontal force distribution with varying degrees of sophistication. Until methods of vertical and horizontal seismic force distribution are better understood and developed for application to light-frame buildings, the importance of engineering judgment cannot be overstated.

1.8.4 Special Seismic Design Considerations

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

Seismic building code developments have introduced several factors and provisions that attempt to address various problems or uncertainties in the design process. Unfortunately, these factors appear to introduce as many uncertainties as they address. Codes have tended to become more complicated to apply or understand, perhaps taking away some important basic principles in seismic design that, when understood, would provide guidance in the application of engineering judgment. Many of the problems stem from the use of the seismic response modifier R which is a concept first introduced to seismic design codes some time in the 1950s. Some of the issues and concerns are briefly described below.

Also known as "reserve strength," the concept of over-strength is a realization that a shear resisting system's ultimate capacity is usually significantly higher than required by a design load as a result of intended safety margins. At the same time, the seismic ground motion (load) is reduced by the R factor to account for ductile response of the building system, among other things. The actual forces experienced on various components (i.e. connections) during a design level event can be substantially higher, even though the resisting system may be able to effectively dissipate those forces. Over-strength factors have been included in the newer seismic codes with recommendations to assist in designing components that may experience higher forces than determined otherwise for the building lateral force resisting system using methods similar to Equation 1.8.1. Over-strength factors should not be considered an exact by the engineer and that actual values of over-strength can vary substantially.

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

The redundancy factor was postulated to address the reliability of lateral force-resisting systems by encouraging multiple lines of shear resistance in a building. It is now included in some seismic design provisions. Since it appears that redundancy factors have little technical basis and insufficient verification relative to light-frame structures, they are not explicitly addressed in this course. Residential buildings are generally recognized for their inherent redundancies that are systematically overlooked when designating and defining a lateral force resisting system for the purpose of executing a rational design. However, this principle is important to consider. For example, it would not be wise to rely on one or two shear-resisting components to support a building. In most applications of light-frame construction, even a single shear wall line has several individual segments and numerous connections that resist shear forces. At a minimum, there are two such shear wall lines in either orientation of the building, not to mention interior walls and other nonstructural elements that contribute to the redundancy of typical light-frame homes. Redundancy is an area where exact guidance does not exist and the engineer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

Deflection amplification has been used in past and current seismic design codes to adjust the deflection and/or story drift determined by use of the design seismic shear load (as adjusted downward by the R factor) relative to that actually experienced without allowance for modified response (i.e., load not adjusted down by the R factor). For wood-framed shear wall construction, the deflection calculated at the nominal seismic shear load (Equation 1.8.1) is multiplied by a factor of 4. The estimate of deflection or drift of the shear wall (or entire story) based on the design seismic shear load would be increased four-fold. The conditions that lead to this level of deflection amplification and the factors that may affect it in a particular design are not exact (and are not obvious to the engineer). As a result, conservative drift amplification values are usually selected for code purposes. Regardless, deflection or drift calculations are rarely applied in a residential (low-rise) wood-framed building design for the following.

- A methodology is not generally available to predict the drift behavior of light-frame buildings reliably and accurately.
- The current design values used for shear wall design are relatively conservative and are usually assumed to provide adequate stiffness (i.e., limit drift).
- Code required drift limits have not been developed for specific application to light-frame residential construction. Deformation amplification is an area where exact guidance does not exist and predictive tools are unreliable. Therefore, the engineer must exercise reasonable care in accordance with the applicable building code requirements.

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

Irregularities are generally separated into plan and vertical structural irregularities. Plan structural irregularities include torsional imbalances that result in excessive rotation of the building, re-entrant corners creating “wings” of a building, floor or roof diaphragms with large openings or non-uniform stiffness, out-of-plane offsets in the lateral force resistance path, and nonparallel resisting systems. Vertical structural irregularities include stiffness irregularities (i.e., a “soft” story), capacity irregularities (i.e., a “weak” story), weight (mass) irregularity (i.e., a “heavy” story), and geometric discontinuities affecting the interaction of lateral resisting systems on adjacent stories.

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

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

It should be understood that the general objective of current and past seismic building code provisions has been to prevent collapse in extreme seismic events such that "protection of life is reasonably provided, but not with complete assurance. It is believed that damage can be controlled by use of a smaller R factor or a larger safety factor.

It has also been suggested using a higher design event. Either approach may indirectly reduce damage or improve performance. It does not necessarily improve the predictability of building performance and may have uncertain benefits in many cases. Some practical considerations as discussed above may lead to better performing buildings, at least from the perspective of controlling damage.

1.9 Other Load Conditions

In addition to the loads covered in Sections 1.3 through 1.8 that are typically considered in the design of a home, other "forces of nature" may create loads on buildings. Some examples include

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

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

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

As noted earlier, this course does not address loads from flooding, ice, rain, and other exceptional sources. The engineer should refer to other resources for information regarding special load conditions.

Design Examples

EXAMPLE 1.1 Design Gravity Load Calculations and Use of ASD Load Combinations

Given

- Three-story conventional wood-framed home
- 28' x 44' plan, clear-span roof, floors supported at mid-span
- Roof dead load = 15 psf (Table 1.2)
- Wall dead load = 8 psf (Table 1.2)
- Floor dead load = 10 psf (Table 1.2)
- Roof snow load = 16 psf (Section 1.7)
- Attic live load = 10 psf (Table 1.4)
- Second- and third-floor live load = 30 psf (Table 1.4)
- First-floor live load = 40 psf (Table 1.4)

Find

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

Solution
1. Gravity load on first-story exterior bearing wall

- Determine loads on wall

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

- Apply applicable ASD load combinations (Table 1.1)

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

Wall axial gravity load = 478 plf + 420 plf + 0.3 (224 plf)
= 965 plf*
*equals 1,055 plf if full attic live load allowance is included with L

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

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

Load condition (a) controls the gravity load analysis for the bearing wall. The same load applies to the design of headers as well as to the wall studs. Of course, combined lateral (bending) and axial loads on the wall studs also need to be checked (i.e., D+W); refer to Table 1.1 and Example 1.2. For nonload-bearing exterior walls (i.e., gable-end curtain walls), contributions from floor and roof live loads may be negligible (or significantly reduced), and the D+W load combination likely governs the design.

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

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

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

(b) Live load area reduction (Equation 1.4.1)

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

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

- Apply ASD load combinations (Table 1.1)

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

Note. If LRFD material design specifications are used, the various loads would be factored in accordance with Table 1.1. All other considerations and calculations remain unchanged.

EXAMPLE 1.2 Design Wind Load Calculations and Use of ASD Load Combinations

Given

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

Find

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

Solution

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

Step 1: Velocity pressure = 14.6 psf (Table 1.7)
Step 2: Adjusted velocity pressure = 0.9* x 14.6 psf = 13.1 psf
*adjustment for wind directionality (V<110 mph)
Step 3: Lateral roof coefficient = 0.6 (Table 1.8)
Lateral wall coefficient = 1.2 (Table 1.8)
Step 4: Skip
Step 5: Determine design wind pressures
Wall projected area pressure = (13.1 psf)(1.2) = 15.7 psf
Roof projected area pressure = (13.1 psf)(0.6) = 7.9 psf

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

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

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

Now determine shear load on the first-story end wall

Shear = (roof VPA)(roof projected area pressure) + (wall VPA)(wall projected area pressure)
= (180 ft_2)(7.9 psf) + (286 ft_2)(15.7 psf)
= 5,912 lbs

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

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

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Roof uplift pressure coefficient = -1.0 (Table 1.9)
Roof overhang pressure coefficient = 0.8 (Table 1.9)
Step 5: Determine design wind pressure
Roof horizontal projected area (HPA) pressure = -1.0 (13.1 psf)
= -13.1 psf
Roof overhang pressure = 0.8 (13.1 psf) = 10.5 psf (upward)

Now determine gross uplift at roof-wall reaction

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

Roof dead load reaction = 1/2 (roof span)(uniform dead load)
= 1/2 (30 ft)(15 psf*)
*Table 1.2
= 225 plf (downward)

Now determine net design uplift load at roof-wall connection

Net design uplift load = $0.6D + W_u$ (Table 1.1)
= 0.6 (225 plf) + (-207 plf)
= -54 plf (net uplift)

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

3. Roof sheathing pull-off (suction) pressure

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Roof sheathing pressure coefficient (suction) = -2.2 (Table 1.9)
Step 5: Roof sheathing pressure (suction) = (13.1 psf)(-2.2)
= -28.8 psf

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

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

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

4. Load on a roof truss

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Roof truss pressure coefficient = -0.9, +0.4 (Table 1.9)
Step 5: Determine design wind pressures

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

Since the inward wind pressure is less than the minimum roof live load (i.e., 15 psf, Table 1.4), the following load combinations would govern the roof truss design while the D+W load combination could be dismissed (refer to Table 1.1):

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

5. Load on a rafter

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Rafter pressure coefficient = -1.2, +0.7 (Table 1.9)
Step 5: Determine design wind pressures

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

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

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

Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Wall stud pressure coefficient = -1.2, +1.1 (Table 1.9)
Step 5: Determine design wind pressures

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

Obviously, the outward pressure of 15.7 psf governs the out-of-plane bending load design of the wall stud. Since the load is a lateral pressure (not uplift), the applicable load combination is D+W (refer to Table 1.1), resulting in a combined axial and bending load. The axial load would include the tributary building dead load from supported assemblies (i.e., walls, floors, and roof). The bending load would the be determined by using the wind pressure of 15.7 psf applied to the stud as a uniform line load on a simply supported beam calculated as follows:

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

Of course, the following gravity load combinations would also need to be considered in the stud design (refer to Table 1.1):

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

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

EXAMPLE 1.3 Design Earthquake Load Calculation

Given

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

Find

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

Solution

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

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

Total tributary weight = 52,016 lb

2. Determine total seismic story shear on first story

$S_{DS} = 2/3\ (S_s)(F_a)$ (Equation 1.8.2)
$= 2/3\ (1.0g)(1.1)$ ($F_a = 1.1$ from Table 1.11)
$= 0.74\ g$

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

$$= \frac{1.2(0.74g)}{5.5} (52{,}016 \text{ lb}) \qquad (R = 5.5 \text{ from Table 1.12})$$

$$= 8{,}399 \text{ lb}$$

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

Assume that the building mass is evenly distributed and that stiffness is also reasonably balanced between the two end walls.
With the above assumption, the load is simply distributed to the end walls according to tributary weight (or plan area) of the building. Therefore,

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

Note that the design shear load from wind (100 mph gust, exposure B) in Example 1.2 is somewhat greater (5,912 lbs).

Appendix A

Unit Conversions

The following list provides the conversion relationship between U.S. customary units and the International System (SI) units. A complete guide to the SI system and its use can be found in ASTM E 380, Metric Practice.

To convert from	to	multiply by
Length		
inch (in.)	meter(μ)	25,400
inch (in.)	centimeter	2.54
inch (in.)	meter(m)	0.0254
foot (ft)	meter(m)	0.3048
yard (yd)	meter(m)	0.9144
mile (mi)	kilometer(km)	1.6
Area		
square foot (sq ft)	square meter(sq m)	0.09290304
square inch (sq in)	square centimeter(sq cm)	6.452
square inch (sq in.)	square meter(sq m)	0.00064516
square yard (sq yd)	square meter(sq m)	0.8391274
square mile (sq mi)	square kilometer(sq km)	2.6
Volume		
cubic inch (cu in.)	cubic centimeter(cu cm)	16.387064
cubic inch (cu in.)	cubic meter(cu m)	0.00001639
cubic foot (cu ft)	cubic meter(cu m)	0.02831685
cubic yard (cu yd)	cubic meter(cu m)	0.7645549
gallon (gal) Can. liquid	liter	4.546
gallon (gal) Can. liquid	cubic meter(cu m)	0.004546
gallon (gal) U.S. liquid*	liter	3.7854118
gallon (gal) U.S. liquid	cubic meter(cu m)	0.00378541
fluid ounce (fl oz)	milliliters(ml)	29.57353
fluid ounce (fl oz)	cubic meter(cu m)	0.00002957
Force		
kip (1000 lb)	kilogram (kg)	453.6
kip (1000 lb)	Newton (N)	4,448.222
pound (lb)	kilogram (kg)	0.4535924
pound (lb)	Newton (N)	4.448222
Stress or pressure		
kip/sq inch (ksi)	megapascal (Mpa)	6.894757
kip/sq inch (ksi)	kilogram/square centimeter (kg/sq cm)	70.31

Appendix A - Unit Conversions

To convert from	to	multiply by
pound/sq inch (psi)	kilogram/square centimeter (kg/sq cm)	0.07031
pound/sq inch (psi)	pascal (Pa) *	6,894.757
pound/sq inch (psi)	megapascal (Mpa)	0.00689476
pound/sq foot (psf)	kilogram/square meter (kg/sq m)	4.8824
pound/sq foot (psf)	pascal (Pa)	47.88
Mass (weight)		
pound (lb) avoirdupois	kilogram (kg)	0.4535924
ton, 2000 lb	kilogram (kg)	907.1848
grain	kilogram (kg)	0.0000648
Mass (weight) per length)		
kip per linear foot (klf)	kilogram per meter (kg/m)	0.001488
pound per linear foot (plf)	kilogram per meter (kg/m)	1.488
Moment		
1 foot-pound (ft-lb)	Newton-meter (N-m)	1.356
Mass per volume (density)		
pound per cubic foot (pcf)	kilogram per cubic meter (kg/cu m)	16.01846
pound per cubic yard (lb/cu yd)	kilogram per cubic meter (kg/cu m)	0.5933
Velocity		
mile per hour (mph)	kilometer per hour (km/hr)	1.60934
mile per hour (mph)	kilometer per second (km/sec)	0.44704
Temperature		
degree Fahrenheit (°F)	degree Celsius (°C)	$t_C = (t_F-32)/1.8$
degree Fahrenheit (°F)	degree Kelvin (°K)	$t_K = (t_F + 459.7)/1.8$
degree Kelvin (°F)	degree Celsius (°C)	$t_C = (t_K - 32)/1.8$

*One U.S. gallon equals 0.8327 Canadian gallon
**A pascal equals 1000 Newton per square meter.

The prefixes and symbols below are commonly used to form names and symbols of the decimal multiples and submultiples of the SI units.

Multiplication Factor	Prefix	Symbol
$1{,}000{,}000{,}000 = 10^9$	giga	G
$1{,}000{,}000 = 10^6$	mega	M
$1{,}000 = 10^3$	kilo	k
$0.01 = 10^{-2}$	centi	c
$0.001 = 10^{-3}$	milli	m
$0.000001 = 10^{-6}$	micro	µ
$0.000000001 = 10^{-9}$	nano	n