
SECTION TEN

WOOD CONSTRUCTION

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Wood is the only renewable source for building materials. It comes from forests that are continually being replanted as they are harvested. This practice ensures a plentiful supply of wood for construction and for a myriad of other uses.

Compared to other building materials, wood has a very high ratio of strength to weight. This makes it very economical for use in all types of construction. Wood also has an aesthetic quality and natural warmth unequalled by other building materials.

Wood has inherent characteristics with which construction users should be familiar. For example, as a consequence of its biological origin, it is nonhomogeneous. Also, properties of pieces of wood from different species of tree may be considerably different, and even properties of pieces of wood from the same tree may differ. In the past, determination of engineering properties depended heavily on visual inspection, keyed to averages, of wood pieces. Research, however, has made possible better estimates of these properties. It is no longer necessary to rely so heavily on visual inspection. Greater accuracy in determination of engineering properties has been made possible by mechanical grading procedures.

Improvements in adhesives for wood also have contributed to the betterment of wood construction. These advances in adhesion technology combined with a desire to utilize more efficiently available wood-fiber resources have led to increasing use of such products as oriented strand board (OSB), glued-laminated timber (glulam), wood I joists, and structural composite lumber (SCL).

10.1 BASIC CHARACTERISTICS OF WOOD

Wood differs in several significant ways from other building materials. Its cellular structure is responsible, to a considerable degree, for these differences. Because of

TABLE 10.1 Shrinkage Values of Wood Based on Dimensions When Green

Species	Dried to 20% MC*			Dried to 6% MC†			Dried to 0% MC		
	Radial, %	Tangential, %	Volumetric, %	Radial, %	Tangential, %	Volumetric, %	Radial, %	Tangential, %	Volumetric, %
Softwood:‡									
Cedar:									
Alaska	0.9	2.0	3.1	2.2	4.8	7.4	2.8	6.0	9.2
Incense	1.1	1.7	2.5	2.6	4.2	6.1	3.3	5.2	7.6
Port Orford	1.5	2.3	3.4	3.7	5.5	8.1	4.6	6.9	10.1
Western red	0.8	1.7	2.3	1.9	4.0	5.4	2.4	5.0	6.8
Cypress, southern	1.3	2.1	3.5	3.0	5.0	8.4	3.8	6.2	10.5
Douglas fir:									
Coast region	1.7	2.6	3.9	4.0	6.2	9.4	5.0	7.8	11.8
Inland region	1.4	2.5	3.6	3.3	6.1	8.7	4.1	7.6	10.9
Rocky Mountain	1.2	2.1	3.5	2.9	5.0	8.5	3.6	6.2	10.6
Fir, white	1.1	2.4	3.3	2.6	5.7	7.8	3.2	7.1	9.8
Hemlock:									
Eastern	1.0	2.3	3.2	2.4	5.4	7.8	3.0	6.8	9.7
Western	1.4	2.6	4.0	3.4	6.3	9.5	4.3	7.9	11.9
Larch, western	1.4	2.7	4.4	3.4	6.5	10.6	4.2	8.1	13.2
Pine:									
Eastern white	0.8	2.0	2.7	1.8	4.8	6.6	2.3	6.0	8.2
Lodgepole	1.5	2.2	3.8	3.6	5.4	9.2	4.5	6.7	11.5
Norway	1.5	2.4	3.8	3.7	5.8	9.2	4.6	7.2	11.5
Ponderosa	1.3	2.1	3.2	3.1	5.0	7.7	3.9	6.3	9.6
Southern (avg.)	1.6	2.6	4.1	4.0	6.1	9.8	5.0	7.6	12.2
Sugar	1.0	1.9	2.6	2.3	4.5	6.3	2.9	5.6	7.9
Western white	1.4	2.5	3.9	3.3	5.9	9.4	4.1	7.4	11.8
Redwood (old growth)	0.9	1.5	2.3	2.1	3.5	5.4	2.6	4.4	6.8
Spruce:									
Engelmann	1.1	2.2	3.5	2.7	5.3	8.3	3.4	6.6	10.4
Sitka	1.4	2.5	3.8	3.4	6.0	9.2	4.3	7.5	11.5

TABLE 10.1 Shrinkage Values of Wood Based on Dimensions When Green (Continued)

Species	Dried to 20% MC*			Dried to 6% MC†			Dried to 0% MC		
	Radial, %	Tangential, %	Volumetric, %	Radial, %	Tangential, %	Volumetric, %	Radial, %	Tangential, %	Volumetric, %
Hardwoods:‡									
Ash, white	1.6	2.6	4.5	3.8	6.2	10.7	4.8	7.8	13.4
Beech, American	1.7	3.7	5.4	4.1	8.8	13.0	5.1	11.0	16.3
Birch:									
Sweet	2.2	2.8	5.2	5.2	6.8	12.5	6.5	8.5	15.6
Yellow	2.4	3.1	5.6	5.8	7.4	13.4	7.2	9.2	16.7
Elm, rock	1.6	2.7	4.7	3.8	6.5	11.3	4.8	8.1	14.1
Gun, red	1.7	3.3	5.0	4.2	7.9	12.0	5.2	9.9	15.0
Hickory:									
Pecan§	1.6	3.0	4.5	3.9	7.1	10.9	4.9	9.8	13.6
True	2.5	3.8	6.0	6.0	9.0	14.3	7.5	11.3	17.9
Maple, hard	1.6	3.2	5.0	3.9	7.6	11.9	4.9	9.5	14.9
Oak:									
Red	1.3	2.7	4.5	3.2	6.6	10.8	4.0	8.2	13.5
White	1.8	3.0	5.3	4.2	7.2	12.6	5.3	9.0	15.8
Poplar, yellow	1.3	2.4	4.1	3.2	5.7	9.8	4.0	7.1	12.3

*MC = moisture content, as a percent of oven-dry wood. These shrinkage values have been taken as one-third the shrinkage to the oven-dry condition as given in the last three columns of this table.

†These shrinkage values have been taken as four-fifths of the shrinkage to the oven-dry condition as given in the last columns of this table.

‡The total longitudinal shrinkage of normal species from fiber saturation to oven-dry condition is minor. It usually ranges from 0.17 to 0.3% of the green dimensions.

§ Average of butternut hickory, nutmeg hickory, water hickory, and pecan.

this structure, structural properties depend on grain orientation. While most structural materials are essentially isotropic, with nearly equal properties in all directions, wood has three principal grain directions—longitudinal, radial, and tangential. Loading in the longitudinal direction is referred to as parallel to the grain, whereas transverse loading is considered to be across the grain. Parallel to the grain, wood possesses high strength and stiffness characteristics. Across the grain, strength and stiffness are much lower. In tension, wood stressed parallel to the grain is 25 to 40 times stronger than when stressed across the grain. In compression, wood loaded parallel to the grain is 6 to 10 times stronger than when loaded perpendicular to the grain. Furthermore, a wood member has three moduli of elasticity, with a ratio of largest to smallest as large as 150:1.

Wood undergoes dimensional changes from causes different from those in most other structural materials. For instance, thermal expansion of wood is so small as to be unimportant in ordinary usage. Significant dimensional changes, however, occur because of gain or loss in moisture. Swelling and shrinkage caused by moisture changes vary in the three grain directions; these size changes are about 5 to 11% tangentially, 3 to 7% radially, but only 0.1 to 0.3% longitudinally. Table 10.1 gives shrinkage values for some commonly used species of wood.

Wood offers numerous advantages in construction applications—warmth and beauty, versatility, durability, workability, low cost per pound, high strength-to-weight ratio, good electrical insulation, low thermal conductance, and excellent strength at low temperatures. It has high shock-absorption capacity. It can withstand good wearing qualities, particularly on end grain. It can be bent easily to relatively sharp curvatures. A wide range of finishes can be applied for decorative or protective purposes. Wood can be used in both wet and dry applications. Preservative treatments are available for use when necessary, as are fire retardants (not appropriate for all wood products). Also, there is a choice of a wide range of species with a range of unique properties.

In addition, a wide variety of wood framing systems is available. The intended use of a structure, geographical location, configuration required, cost, and many other factors determine the best framing system to be used for a particular project.

Wood is naturally resistant to many chemicals that are highly corrosive to other materials. It is superior to many building materials in resistance to mild acids, particularly at ordinary temperatures. It has excellent resistance to most organic acids, notably acetic. However, wood is seldom used in contact with solutions that are more than weakly alkaline. Oxidizing chemicals and solutions of iron salts, in combination with damp conditions, should be avoided.

Wood is composed of roughly 50 to 70% cellulose, 25 to 30% lignin, and 5% extractives with less than 2% protein. Acids such as acetic, formic, lactic, and boric do not ionize sufficiently at room temperature to attack cellulose, and thus do not harm wood.

When the pH of aqueous solutions of weak acids is 2 or more, the rate of hydrolysis of cellulose is small and dependent on the temperature. A rough approximation of this temperature effect is that, for every 20°F increase, the rate of hydrolysis doubles. Acids with pH values above 2, or bases with pH below 10, have little weakening effect on wood at room temperature, if the duration of exposure is moderate.

Design Recommendations. The following recommendations aim at achieved economical designs with wood framing:

Use standard sizes and grades of lumber. Consider using standardized structural components, whether lumber, stock glued-laminated beams, or other framing members designed for structural adequacy, efficiency, and economy.

TABLE 10.2 Typical Span Range for Main Wood Framing Members

Framing member	Typical span range, ft
Roof beams	
Simple span:	
Constant depth	
Solid-sawn	10–25
SCL	10–40
Glulam	20–100
Tapered glulam	25–100
Double-tapered pitched and curved glulam beams	25–100
Curved glulam beams	25–100
Simple beam with overhangs (overhangs typically limited to 25% of main span)	
Solid-sawn	10–25
Glulam	10–100
Continuous span:	
Solid-sawn	10–25
Glulam	10–50
Glulam arches:	
Three-hinged:	
Gothic	40–100
Tudor	40–120
A-frame	20–100
Three-centered	40–250
Parabolic	40–250
Radial	40–250
Two-hinged	
Radial	50–200
Parabolic	50–200
Roof trusses:	
Flat or parallel chord	25–100
Triangular or pitched	25–100
Bowstring	50–200
Domes	200–500
Simple-span floor beams:	
Solid sawn	6–20
Glulam	10–40
SCL	10–40
Continuous floor beams	10–40
Roof sheathing and decking:	
1-in sheathing	1–4
2-in sheathing	6–10
3-in roof deck	8–15
4-in roof deck	12–20

* Not applicable.

Use standard details wherever possible. Avoid specially designed and manufactured connecting hardware.

Use as simple and as few joints as possible. Place splices, when required, in areas of lowest stress. Do not locate splices where bending moments are large, thus avoiding design, installation, and fabrication difficulties.

Avoid unnecessary variations in cross section of members along their length.

Use identical member designs repeatedly throughout a structure, whenever practicable. Keep the number of different arrangements to a minimum.

Specify required design stresses to permit the widest range of products that can be used for a given design situation.

Use wood products pressure treated with preservatives where service conditions dictate. Such treatment need not be used where decay or insect attack hazards do not exist. Fire-retardant treatments may be used to meet a specific flame-spread rating for interior finish, but are not necessary for large-cross-sectional members that are widely spaced and have a natural resistance to fire because of their relatively large size.

Instead of long, simple spans, consider using continuous or suspended spans or simple spans with overhangs.

Select an appearance grade best suited to the project. Do not specify the highest quality appearance grade available for all members if it is not required.

Table 10.2 may be used as a general guide to typical ranges of spans for roof and main floor framing members (excluding repetitive member joist and rafter applications).

10.2 SECTIONAL PROPERTIES OF WOOD PRODUCTS

Dressed sizes of sawn lumber are given in the grading rules of agencies that formulate and maintain such rules and in Table 10.3. The nominal and dressed sizes are developed in accordance with the American Softwood Lumber Standard, Voluntary Product Standard PS 20-94. These sizes are generally available, but it is good practice to consult suppliers before specifying sizes not commonly used to find out what sizes are on hand or can be readily secured.

The supplement to the “National Design Specification® for Wood Construction” published by the American Forest & Paper Association (formerly the National Forest Products Association) presents tables of section properties of standard dressed sawn lumber and glulam timber. Standard finished sizes of structural glulam timber should be used to the extent that conditions permit. These standard finished sizes are based on lumber sizes given in Voluntary Product Standard PS 20-94. Other finished sizes may be used to meet the size requirements of a design, or to meet other special requirements.

For the manufacture of glulam nominal 2-in-thick lumber, surfaced to $1\frac{3}{8}$ in or $1\frac{1}{2}$ in before gluing, is used to laminate straight members and curved members having radii of curvature within the bending-radius limitations for the species. Nominal 1-in-thick lumber, surfaced to $\frac{3}{4}$ in before gluing, may be used for laminating

TABLE 10.3 Nominal and Minimum Dressed Sizes of Sawn Lumber

Item	Thickness, in			Face width, in			
	Nominal	Minimum dressed		Nominal	Minimum dressed		
		Dry*	Green†		Dry*	Green†	
Boards	1	$\frac{3}{4}$	$\frac{25}{32}$	2	$1\frac{1}{2}$	$1\frac{1}{16}$	
	$1\frac{1}{4}$	1	$1\frac{1}{32}$	3	$2\frac{1}{2}$	$2\frac{9}{16}$	
	$1\frac{1}{2}$	$1\frac{1}{4}$	$1\frac{9}{32}$	4	$3\frac{1}{2}$	$3\frac{9}{16}$	
				5	$4\frac{1}{2}$	$4\frac{5}{8}$	
				6	$5\frac{1}{2}$	$5\frac{5}{8}$	
				7	$6\frac{1}{2}$	$6\frac{5}{8}$	
				8	$7\frac{1}{4}$	$7\frac{1}{2}$	
				9	$8\frac{1}{4}$	$8\frac{1}{2}$	
				10	$9\frac{1}{4}$	$9\frac{1}{2}$	
				11	$10\frac{1}{4}$	$10\frac{1}{2}$	
				12	$11\frac{1}{4}$	$11\frac{1}{2}$	
				14	$13\frac{1}{4}$	$13\frac{1}{2}$	
				16	$15\frac{1}{4}$	$15\frac{1}{2}$	
	Dimension Lumber	2	$1\frac{1}{2}$	$1\frac{9}{16}$	2	$1\frac{1}{2}$	$1\frac{9}{16}$
		$2\frac{1}{2}$	2	$2\frac{1}{16}$	3	$2\frac{1}{2}$	$2\frac{9}{16}$
		3	$2\frac{1}{2}$	$2\frac{9}{16}$	4	$3\frac{1}{2}$	$3\frac{9}{16}$
$3\frac{1}{2}$		3	$3\frac{1}{16}$	5	$4\frac{1}{2}$	$4\frac{5}{8}$	
4		$3\frac{1}{2}$	$3\frac{9}{16}$	6	$5\frac{1}{2}$	$5\frac{5}{8}$	
$4\frac{1}{2}$		4	$4\frac{1}{16}$	8	$7\frac{1}{4}$	$7\frac{1}{2}$	
				10	$9\frac{1}{4}$	$9\frac{1}{2}$	
				12	$11\frac{1}{4}$	$11\frac{1}{2}$	
Timbers				14	$13\frac{1}{4}$	$13\frac{1}{2}$	
				16	$15\frac{1}{4}$	$15\frac{1}{2}$	
	5 and thicker		$\frac{1}{2}$ in less	5 and wider		$\frac{1}{2}$ in less	

*Dry lumber is defined as lumber seasoned to a moisture content of 19% or less.

†Green lumber is defined as lumber having a moisture content in excess of 19%.

curved members when the bending radius is too tight to permit use of nominal 2-in-thick laminations. Other lamination thicknesses may be used to meet special curving requirements.

Standard sizes and grades of structural panels are given in U.S. Product Standard PS 1-95 for Construction and Industrial Plywood and "Performance Standard for Wood-Based Structural-Use Panels," Voluntary Product Standard, PS 2-92. See also Art. 10.12.

Weight and Specific Gravity. Specific gravity is a reliable indicator of fiber content. Also, specific gravity and the strength and stiffness of solid wood or laminated products are interrelated. See Table 10.4 for weights and specific gravities of several commercial lumber species.

TABLE 10.4 Weights and Specific Gravities of Commercial Lumber Species

Species	Specific gravity based on oven-dry weight and volume at 12% moisture content	Weight, lb per ft ³			Moisture content when green (avg), %	Specific gravity based on oven-dry weight and volume when green	Weight when green, lb per ft ³
		At 12% moisture content	At 20% moisture content	Adjusting factor for each 1% change in moisture content			
Softwoods:							
Cedar:							
Alaska	0.44	31.1	32.4	0.170	38	0.42	35.5
Incense	0.37	25.0	26.4	0.183	108	0.35	42.5
Port Orford	0.42	29.6	31.0	0.175	43	0.40	35.0
Western red	0.33	23.0	24.1	0.137	37	0.31	26.4
Cypress, southern	0.46	32.1	33.4	0.167	91	0.42	45.3
Douglas fir:							
Coast region	0.48	33.8	35.2	0.170	38	0.45	38.2
Inland region	0.44	31.4	32.5	0.137	48	0.41	36.3
Rocky Mountain	0.43	30.0	31.4	0.179	38	0.40	34.6
Fir, white	0.37	26.3	27.3	0.129	115	0.35	39.6
Hemlock:							
Eastern	0.40	28.6	29.8	0.150	111	0.41	43.4
Western	0.42	29.2	30.2	0.129	74	0.38	37.2
Larch, western	0.55	38.9	40.2	0.170	58	0.51	46.7
Pine:							
Eastern white	0.35	24.9	26.2	0.167	73	0.36	35.1
Lodgepole	0.41	28.8	29.9	0.142	65	0.38	36.3
Norway	0.44	31.0	32.1	0.142	92	0.41	42.3
Ponderosa	0.40	28.1	29.4	0.162	91	0.38	40.9
Southern shortleaf	0.51	35.2	36.5	0.154	81	0.46	45.9
Southern longleaf	0.58	41.1	42.5	0.179	63	0.54	50.2
Sugar	0.36	25.5	26.8	0.162	137	0.35	45.8
Western white	0.38	27.6	28.6	0.129	54	0.36	33.0

TABLE 10.4 Weights and Specific Gravities of Commercial Lumber Species (Continued)

Species	Specific gravity based on oven-dry weight and volume at 12% moisture content	Weight, lb per ft ³			Moisture content when green (avg), %	Specific gravity based on oven-dry weight and volume when green	Weight when green, lb per ft ³
		At 12% moisture content	At 20% moisture content	Adjusting factor for each 1% change in moisture content			
Redwood	0.40	28.1	29.5	0.175	112	0.38	45.6
Spruce:							
Engelmann	0.34	23.7	24.7	0.129	80	0.32	32.5
Sitka	0.40	27.7	28.8	0.145	42	0.37	32.0
White	0.40	29.1	29.9	0.104	50	0.37	33.0
Hardwoods:							
Ash, white	0.60	42.2	43.6	0.175	42	0.55	47.4
Beech, American	0.64	43.8	45.1	0.162	54	0.56	50.6
Birch:							
Sweet	0.65	46.7	48.1	0.175	53	0.60	53.8
Yellow	0.62	43.0	44.1	0.142	67	0.55	50.8
Elm, rock	0.63	43.6	45.2	0.208	48	0.57	50.9
Gum	0.52	36.0	37.1	0.133	115	0.46	49.7
Hickory:							
Pecan	0.66	45.9	47.6	0.212	63	0.60	56.7
Shagbark	0.72	50.8	51.8	0.129	60	0.64	57.0
Maple, sugar	0.63	44.0	45.3	0.154	58	0.56	51.1
Oak:							
Red	0.63	43.2	44.7	0.187	80	0.56	56.0
White	0.68	46.3	47.6	0.167	68	0.60	55.6
Poplar, yellow	0.42	29.8	31.0	0.150	83	0.40	40.5

10.3 DESIGN VALUES FOR LUMBER AND TIMBER

Design values for an extensive range of sawn lumber and timber are tabulated in the supplement to the “National Design Specification® for Wood Construction,” (NDS), American Forest & Paper Association (AF&PA).

10.3.1 Lumber

Design values for lumber are contained in grading rules established by the National Lumber Grades Authority (Canadian), Northeastern Lumber Manufacturers Association, Northern Softwood Lumber Bureau, Redwood Inspection Service, Southern Pine Inspection Bureau, West Coast Lumber Inspection Bureau, and Western Wood Products Association. The rules and the design values in them have been approved by the Board of Review of the American Lumber Standards Committee. They also have been certified for conformance with U.S. Department of Commerce Voluntary Product Standard PS 20-94 (American Softwood Lumber Standard).

In addition, design values for visually graded lumber may be established in accordance with ASTM D1990, “Standard Practice for Establishing Allowable Properties for Visually-Graded Dimensional Lumber from In-Grade Tests of Full-Size Specimens.” Design values for visually graded timbers, decking, and some species and grades of dimension lumber are based on provisions of “Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber,” ASTM D245. ASTM D245 also specifies adjustments to be made in the strength properties of small clear specimens of wood, as determined in accordance with “Establishing Clear Wood Strength Values,” ASTM D2555, to obtain design values applicable to normal conditions of service. The adjustments account for the effects of knots, slope of grain, splits, checks, size, duration of load, moisture content, and other influencing factors. Lumber structures designed with working stresses derived from D245 procedures and standard design criteria have a long history of satisfactory performance.

Design values for machine stress-rated (MSR) lumber and machine-evaluated lumber (MEL) are based on nondestructive tests of individual wood pieces. Certain visual-grade requirements also apply to such lumber. The stress rating system used for MSR lumber and MEL is checked regularly by the responsible grading agency for conformance with established certification and quality-control procedures.

10.3.2 Glued-Laminated Timber

Design values for glulam timber, developed by the American Institute of Timber Construction (AITC) and Engineered Wood Systems (EWS) in accordance with principles originally established by the U.S. Forest Products Laboratory, are included in the supplement to the NDS. The principles are the basis for the “Standard Method for Establishing Stresses for Structural Glued-Laminated Timber (Glulam),” ASTM D3737 which specifies procedures for obtaining design values that account for the effects of knots, slope of grain, density, size of member, curvature, number of laminations, and other factors unique to glulam. The satisfactory performance of structures made with glulam members conforming to AITC and EWS specifications and manufactured in accordance with “Structured Glued-Laminated Tim-

ber," ANSI A190.1, demonstrates the validity of the methods used to establish glulam design values.

10.4 STRUCTURAL GRADING OF WOOD

Strength properties of wood are closely related to moisture content and specific gravity. Therefore, data on strength properties should be accompanied by corresponding data on these physical properties.

The strength of wood is actually affected by many other factors, including loading rate, load duration, temperature, grain direction, and position of growth rings. Strength is also influenced by inherent growth characteristics, including knots, slope of grain, shakes, and checks. Analysis and integration of available data have yielded a comprehensive set of principles for grading structural lumber (Art. 10.3.1).

The same characteristics that reduce the strength of solid timber also affect the strength of glued-laminated (glulam) members (Art. 10.3.2). There are, however, additional factors peculiar to glulam flexural members that should be considered. For example, knots located near the neutral axis, which is a region of low bending stress, have less effect on strength than knots closer to the outer surfaces, where bending stresses are higher. Thus, strength of a flexural member with low-grade laminations can be improved by substitution of higher grade laminations at the top and bottom of the member.

Dispersement of knots in laminated members has a beneficial effect on strength. With sufficient knowledge of the occurrence of knots within a grade, mathematical estimates of the effect may be established for members containing various numbers of laminations.

10.5 ADJUSTMENT FACTORS FOR STRUCTURAL MEMBERS

Design values obtained by the methods described in Art. 10.3 should be multiplied by adjustment factors based on conditions of use, geometry, and stability. The adjustments are cumulative, unless specifically indicated in the following.

The adjusted design value F'_b for extreme-fiber bending is given by

$$F'_b = F_b C_D C_M C_t C_L C_F C_V C_{fu} C_r C_c C_f \quad (10.1)$$

where F_b = design value for extreme-fiber bending

C_D = load duration factor (Art. 10.5.1)

C_M = wet service factor (Art. 10.5.2)

C_t = temperature factor (Art. 10.5.3)

C_L = beam stability factor (Arts. 10.5.5 and 10.7.2)

C_F = size factor—applicable only to visually graded, sawn lumber and round timber flexural members (Art. 10.5.4)

C_V = volume factor—applicable only to glulam flexural members (Art. 10.5.6)

C_{fu} = flat-use factor—applicable only to dimension-lumber beams 2 to 4 in thick and glulam beams with loads applied parallel to the wide face of the laminations (Art. 10.5.7)

C_r = repetitive member factor—applicable only to dimension-lumber beams 2 to 4 in thick (Art. 10.5.8)

C_c = curvature factor—applicable only to curved portions of glulam beams (Art. 10.5.9)

C_f = form factor (Art. 10.5.10)

For glulam beams, use either C_L or C_V , whichever is smaller, not both, in Eq. (10.1). The adjusted design value for tension F'_t is given by

$$F'_t = F_t C_D C_M C_t C_F \quad (10.2)$$

where F_t = design values for tension.

For shear, the adjusted design value F'_V is computed from

$$F'_V = F_V C_D C_M C_t C_H \quad (10.3)$$

where F_V = design value for shear and C_H = shear stress factor ≥ 1 —permitted for F_V parallel to the grain for sawn lumber members (Art. 10.5.13).

For compression perpendicular to the grain, the adjusted value $F'_{c\perp}$ is obtained from

$$F'_{c\perp} = F_{c\perp} C_M C_t C_b \quad (10.4)$$

where $F_{c\perp}$ = design value for compression perpendicular to the grain and C_b = bearing area factor (Art. 10.5.11).

For compression parallel to the grain, the adjusted design value F'_c is given by

$$F'_c = F_c C_D C_M C_t C_F C_p \quad (10.5)$$

where F_c = design value for compression parallel to grain and C_p = column stability factor (Arts. 10.5.11 and 10.8.1).

For end grain in bearing parallel to the grain, the adjusted design value F'_g is computed from

$$F'_g = F_g C_D C_t \quad (10.6)$$

where F_g = design value for end grain in bearing parallel to the grain. See also Art. 10.11.1.

The adjusted design value for modulus of elasticity E' is obtained from

$$E' = E C_M C_t C_T \quad (10.7)$$

where E = design value for modulus of elasticity

C_T = buckling stiffness factor—applicable only to sawn lumber truss compression chords 2×4 in or smaller, when subject to combined bending and axial compression and plywood sheathing $\frac{3}{8}$ in or more thick is nailed to the narrow face (Art. 10.5.12).

10.5.1 Load Duration Factor

Wood has the capacity to carry substantially greater loads for short periods of time than for long periods. Design values described in Art. 10.3 apply to normal load duration, which is equivalent to application of full design load for a cumulative

duration of about 10 years. The full design load is one that stresses a member to its allowable design value. When the cumulative duration of the full design load differs from 10 years, design values, except $F_{c\perp}$ for compression perpendicular to grain and modulus of elasticity E , should be multiplied by the appropriate load duration factor C_D listed in Table 10.5.

When loads of different duration are applied to a member, C_D for the load of shortest duration should be applied to the total load. In some cases, a larger-size member may be required when one or more of the shorter-duration loads are omitted. Design of the member should be based on the critical load combination. If the permanent load is equal to or less than 90% of the total combined load, the normal load duration will control the design. Both C_D and the modification permitted in design values for load combinations may be used in design.

The duration factor for impact does not apply to connections. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with fire retardants or with waterborne preservatives.

10.5.2 Wet Service Factor

Sawn-lumber design values apply to lumber that will be used under dry service conditions; that is, where moisture content (MC) of the wood will be a maximum of 19% of the oven-dry weight, regardless of MC at time of manufacture. When the MC of sawn lumber or timbers in service will exceed 19% for an extended period of time, design values published in the supplement to the “National Design Specification® for Wood Construction” should be multiplied by the appropriate wet service factor listed in Table 10.6. This reduction factor for timber does not apply to southern pine.

MC of 19% or less is generally maintained in covered structures or in members protected from the weather, including windborne moisture. Wall and floor framing and attached sheathing are usually considered to be such dry applications. These dry conditions are generally associated with an average relative humidity of 80% or less. Framing and sheathing in properly ventilated roof systems are assumed to meet MC criteria for dry conditions of use, even though they are exposed periodically to relative humidities exceeding 80%.

Glulam-timber design values apply when the MC in service is less than 16%, as in most covered structures. When MC of glulam timber under service conditions is 16% or more, design values should be multiplied by the appropriate wet service factor C_M in Table 10.6.

TABLE 10.5 Frequently Used Load Duration Factors C_D

Load duration	C_D	Typical design loads
Permanent	0.9	Dead load
10 years	1.0	Occupancy live load
2 months	1.15	Snow load
7 days	1.25	Construction load
10 minutes	1.6	Wind or seismic load
Impact	2.0	Impact load

TABLE 10.6 Wet Service Factors C_M

Design value	C_M for sawn lumber*	C_M for glulam timber†	C_M for timber
F_b	0.85‡	0.80	1.0
F_t	1.0	0.80	1.0
F_v	0.97	0.875	1.0
$F_{e\perp}$	0.67	0.53	0.67
F_c	0.80§	0.73	0.91
E	0.90	0.833	1.00

* For use where moisture content in service exceeds 19%.

† For use where moisture content in service exceeds 16%.

‡ $C_M = 1.0$ when $F_b C_F \leq 1150$ psi.

§ $C_M = 1.0$ when $F_c C_F \leq 750$ psi.

10.5.3 Temperature Factor

Design values apply to members used in ordinary temperature ranges. (Occasional heating to 150°F is permissible.) Strength properties of wood, however, increase when it is cooled below normal temperatures and decrease when it is heated. Members heated in use to temperatures up to 150°F return essentially to original strength when cooled. Prolonged exposure to temperatures above 150°F, however, may result in permanent loss of strength. Design values for structural members that will experience sustained exposure to elevated temperatures up to 150°F should be multiplied by the appropriate temperature factor C_t listed in Table 10.7.

10.5.4 Size Factor

For visually graded dimension lumber, design values F_b , F_t , and F_c published in the supplement to NDS for all species and species combinations, except southern pine should be multiplied by the appropriate size factor C_F given in Table 10.8 to account for the effects of member size. This factor and the factors used to develop size-specific values for southern pine are based on the adjustment equation given in ASTM D1990. These factors, based on in-grade test data, account for differences in F_b , F_t , and F_c related to width and in F_b and F_t related to length (test span).

For visually graded timbers (5 × 5 in or larger), when the depth d of a stringer, beam, post, or timber exceeds 12 in, the design value for bending for all species should be adjusted by the size factor

TABLE 10.7 Temperature Factors C_t

Design values and in-service moisture conditions	$T \leq 100^\circ\text{F}$	$100^\circ\text{F} < T \leq 125^\circ\text{F}$	$125^\circ\text{F} < T \leq 150^\circ\text{F}$
F_t and E , wet or dry	1.0	0.9	0.9
F_b , F_v , F_c , and $F_{e\perp}$			
Dry	1.0	0.8	0.7
Wet	1.0	0.7	0.5

TABLE 10.8 Size Factors C_F for Sawn Lumber

F_b				F_t	F_c
Grades	Width, in	Thickness, in			
		2 and 3	4		
Select Structural, No. 1 and better, No. 1, No. 2, No. 3	2, 3, and 4	1.5	1.5	1.5	1.15
	5	1.4	1.4	1.4	1.1
	6	1.3	1.3	1.3	1.1
	8	1.2	1.3	1.2	1.05
	10	1.1	1.2	1.1	1.0
	12	1.0	1.1	1.0	1.0
	14 and wider	0.9	1.0	0.9	0.9
Stud	2, 3 and 4	1.1	1.1	1.1	1.05
	5 and 6	1.0	1.0	1.0	1.0
Construction and Standard	2, 3 and 4	1.0	1.0	1.0	1.0
Utility	4	1.0	1.0	1.0	1.0
	2 and 3	0.4		0.4	0.6

$$C_F = (12/d)^{1/9} \quad (10.8)$$

10.5.5 Beam Stability Factor

Design values F_b for bending should be adjusted by multiplying by the beam stability factor C_L specified in Art. 10.7.2. For glulam beams, the smaller value of C_L and the volume factor C_V should be used, not both. See also Art. 10.5.6.

10.5.6 Volume Factor

Design values for bending F_b for glulam beams should be adjusted for the effects of volume by multiplying by

$$C_V = K_L \left[\left(\frac{21}{L} \right) \left(\frac{12}{d} \right) \left(\frac{5.125}{b} \right) \right]^{1/x} \quad (10.9)$$

where L = length of beam between inflection points, ft

d = depth, in, of beam

b = width, in, of beam

= width, in, or widest piece in multiple piece layups with various widths (thus, $b \leq 10.75$ in)

x = 20 for southern pine

= 10 for other species

K_L = loading condition coefficient (Table 10.9)

For glulam beams, the smaller of C_V and the beam stability factor C_L should be used, not both.

TABLE 10.9 Loading Condition Coefficient K_L for Glulam Beams

Single-span beams	
Loading condition	K_L
Concentrated load at midspan	1.09
Uniformly distributed load	1.0
Two equal concentrated loads at third points of span	0.96
Continuous beams of cantilevers	
All loading conditions	1.0

10.5.7 Flat-Use Factor

Design values for sawn lumber beams adjusted by the size factor C_{fu} assume that load will be applied to the narrow face. When load is applied to the wide face (flatwise) of dimension lumber, design values should be multiplied by the appropriate flat-use factor given in Table 10.10. These factors are based on the size-adjustment equation in ASTM D245. Available test results indicate that this equation yields conservative values of C_{fu} .

When a glulam member is loaded parallel to the wide face of the laminations and the member dimension parallel to the face is less than 12 in, the design value for bending for such loading should be multiplied by the appropriate flat-use factor in Table 10.11.

10.5.8 Repetitive Member Factor

Design values for bending F_b may be increased when three or more members are connected so that they act as a unit. The members may be in contact or spaced up to 24 in c to c if joined by transverse load-distributing elements that ensure action of the assembly as a unit. The members may be any piece of dimension lumber subjected to bending, including studs, rafters, truss chords, joists, and decking.

When the criteria are satisfied, the design value for bending of dimension lumber 2 to 4 in thick may be multiplied by the repetitive member factor $C_r = 1.15$.

TABLE 10.10 Flat-Use Factors C_{fu} for Dimension Lumber

Width, in	Thickness, in	
	2 and 3	4
2 and 3	1.0	
4	1.1	1.0
5	1.1	1.05
6	1.15	1.05
8	1.15	1.05
10 and wider	1.2	1.1

TABLE 10.11 Flat-Use Factors C_{fu} for Glulam Beams

Lamination width, in	C_{fu}
$10^{3/4}$ or $10^{1/2}$	1.01
$8^{3/4}$ or $8^{1/2}$	1.04
$6^{3/4}$	1.07
$5^{1/8}$ or 5	1.10
$3^{1/8}$ or 3	1.16
$2^{1/2}$	1.19

This factor applies to three or more essentially parallel members of equal size and with the same orientation that are in direct contact with each other. Transverse connecting elements may be mechanical fasteners, such as through nails, nail gluing, tongue-and-groove joints, or bearing plates, that ensure that the members act together to resist applied bending moments.

For spaced members, the transverse distributing elements should be acceptable to the applicable regulatory agency and should be capable, as demonstrated by test, analysis, or experience, of transmitting design loads without unacceptable deflections or indications of structural weakness. The load may be uniform or concentrated, or both, applied on the surface of the distributing element.

A transverse element attached to the underside of framing members and supporting no uniform load other than its own weight and other incidental light loads, such as insulation, qualifies as a load-distributing element only for bending moment associated with its own weight and that of the framing members to which it is attached. Qualifying construction includes subflooring, finish flooring, exterior and interior wall finish, and cold-formed metal siding with or without backing. Such elements should be fastened to the framing members by approved means, such as nails, glue, staples, or snap-lock joints.

Individual members in a qualifying assembly made of different species or grades are each eligible for the repetitive-member increase in F_b if they satisfy all the preceding criteria.

10.5.9. Curvature Factor and Radial Stresses

For the curved portions of glulam beams, the design value for bending should be multiplied by the curvature factor

$$C_c = 1 - 2000(t/R)^2 \quad (10.10)$$

where t = lamination thickness, in, and R = radius of curvature, in, of inside face of lamination. t/R should not exceed 0.01 for hardwoods and southern pine or 0.008 for other softwoods. The curvature factor does not apply to design values of F_b for the straight portions of a member, regardless of curvature elsewhere.

Radial Tension or Compression. The radial stress induced by a bending moment in a member of constant cross section may be computed from

$$f_r = \frac{3M}{2Rbd} \quad (10.11)$$

where M = bending moment, in-lb

R = radius of curvature at centerline of member, in

b = width of cross section, in

d = depth of cross section, in

When M is in the direction tending to decrease curvature (increase the radius), tensile stresses occur across the grain. For this condition, the allowable tensile stress across the grain is limited to one-third the allowable unit stress in horizontal shear for southern pine for all load conditions, and for Douglas fir and larch for wind or earthquake loadings. The limit is 15 psi for Douglas fir and larch for other types

of loading. These values are subject to modification for duration of load. If these values are exceeded, mechanical reinforcement sufficient to resist all radial tensile stresses is required.

When M is in the direction tending to increase curvature (decrease the radius), the stress is compressive across the grain. For this condition, the allowable stress is limited to that for compression perpendicular to grain for all species.

(K. F. Faherty and T. G. Williamson, "Wood Engineering and Construction Handbook," and D. E. Breyer, "Design of Wood Structures," 2d ed., McGraw-Hill Publishing Company, New York.)

10.5.10 Form Factor

Design values for bending F_b for beams with a circular cross section may be multiplied by a form factor $C_f = 1.18$. For a flexural member with a square cross section loaded in the plane of the diagonal (diamond-shape cross section), C_f may be taken as 1.414.

These form factors ensure that a circular or diamond-shape flexural member has the same moment capacity as a square beam with the same cross-sectional area. If a circular member is tapered, it should be treated as a beam with variable cross section.

10.5.11 Column Stability and Bearing Area Factors

Design values for compression parallel to the grain F_c should be multiplied by the column stability factor C_p specified in Art. 10.8.1.

Design values for compression perpendicular to the grain $F_{c\perp}$ apply to bearing surfaces of any length at the ends of a member and to all bearings 6 in or more long at other locations. For bearings less than 6 in long and at least 3 in from the end of a member, $F_{c\perp}$ may be multiplied by the bearing area factor

$$C_b = \frac{L_b + 0.375}{L_b} \quad (10.12)$$

where L_b = bearing length, in, measured parallel to grain. Equation (10.12) yields the values of C_b for elements with small areas, such as plates and washers, listed in Table 10.12. For round bearing areas, such as washers, L_b should be taken as the diameter.

10.5.12 Buckling Stiffness Factor

The buckling stiffness of a truss compression chord of sawn lumber subjected to combined flexure and axial compression under dry service conditions may be in-

TABLE 10.12 Bearing Area Factors C_b

Bearing length, in	0.50	1.00	1.50	2.00	3.00	4.00	6 or more
Bearing area factor	1.75	1.38	1.25	1.19	1.13	1.10	1.00

creased if the chord is 2×4 in or smaller and has the narrow face braced by nailing to wood structural panel sheathing at least $\frac{3}{8}$ in thick in accordance with good nailing practice. The increased stiffness may be accounted for by multiplying the design value of the modulus of elasticity E by the buckling stiffness factor C_T in column stability calculations. When the effective column length L_e , in, is 96 in or less, C_T may be computed from

$$C_T = 1 + \frac{K_M L_e}{K_T E} \quad (10.13)$$

where $K_M = 2300$ for wood seasoned to a moisture content of 19% or less at time of sheathing attachment
 $= 1200$ for unseasoned or partly seasoned wood at time of sheathing attachment
 $K_T = 0.59$ for visually graded lumber
 $= 0.75$ for machine evaluated lumber (MEL)
 $= 0.82$ for products with a coefficient of variation of 0.11 or less

When L_e is more than 96 in, C_T should be calculated from Eq. (10.13) with $L_e = 96$ in. For additional information on wood trusses with metal-plate connections, see design standards of the Truss Plate Institute, Madison, Wis.

10.5.13 Shear Stress Factor

For dimension-lumber grades of most species or combinations of species, the design value for shear parallel to the grain F_V is based on the assumption that a split, check, or shake that will reduce shear strength 50% is present (Art. 4.34). Reductions exceeding 50% are not required inasmuch as a beam split lengthwise at the neutral axis will still resist half the bending moment of a comparable unsplit beam. Furthermore, each half of such a fully split beam will sustain half the shear load of the unsplit member. The design value F_V may be increased, however, when the length of split or size of check or shake is known and is less than the maximum length assumed in determination of F_V , if no increase in these dimensions is anticipated. In such cases, F_V may be multiplied by a shear stress factor C_H greater than unity.

In most design situations, C_H cannot be applied because information on length of split or size of check or shake is not available. The exceptions, when C_H can be used, include structural components and assemblies manufactured fully seasoned with control of splits, checks, and shakes when the products, in service, will not be exposed to the weather. C_H also may be used in evaluation of the strength of members in service. The "National Design Specifications for Wood Construction," American Forest & Paper Association, lists values of C_H for lumber and timber of various species.

10.6 PRESSURE-PRESERVATIVE TREATMENTS FOR WOOD

Wood members are considered to be permanent without pressure treatment if located in enclosed buildings where good roof coverage, proper roof maintenance,

good joint details, adequate flashing, good ventilation, and a well-drained site assure moisture content of the wood continuously below 20%.

Where wood is in contact with the ground or with water, where there is air and the wood may be alternately wet and dry, a preservative treatment, applied by a pressure process, is necessary to obtain an adequate service life. In enclosed buildings where moisture given off by wet-process operations maintains equilibrium moisture contents in the wood above 19%, wood structural members must be preservatively treated. So must wood exposed outdoors without protective roof covering and where the wood moisture content can go above 19% for repeated or prolonged periods.

Where wood structural members are subject to condensation by being in contact with masonry or concrete, preservative treatment may be necessary.

Design values for wood structural members apply to products pressure-treated by an approved process and with an approved preservative. (The "AWPA Book of Standards," American Wood Preservers Association, Granbury, TX, describes these approved processes.) Design values for pressure-preservative treated lumber are modified with the usual adjustment factors described in Art. 10.5 with one exception. Load duration factors greater than 1.6 (Table 10.5) do not apply to structural members pressure treated with waterborne preservatives or to structural members treated with fire-retardant chemicals.

Each type of preservative and method of treatment has certain advantages. The preservative to be used depends on the service expected of the member for the specific conditions of exposure. The minimum retentions given in the applicable American Wood Preservers Association (AWPA) standards for specific products and end-use applications may be increased where severe climatic or exposure conditions are involved.

Creosote and creosote solutions have low volatility. They are practically insoluble in water, and thus are most suitable for severe exposure, contact with ground or water, and where painting is not a requirement or a creosote odor is not objectionable.

Oilborne chemicals are organic compounds dissolved in an approved petroleum carrier oil, and are suitable for outdoor exposure or where leaching may be a factor, or where painting is not required. Depending on the type of oil used, they may result in relatively clean surfaces. While there is a slight odor from such treatment, it is usually not objectionable.

Waterborne inorganic salts are dissolved in water or aqua ammonia, which evaporates after treatment and leaves the chemicals in the wood. The strength of solutions varies to provide net retention of dry salt required. These salts are suitable where clean and odorless surfaces are required. The surfaces are paintable after proper seasoning. See also Art. 4.36.

("Design of Wood-Frame Structures for Permanence," WCD No. 6, American Forest & Paper Association, Washington, D.C.)

Fire-retardant treatment with approved chemicals can make wood highly resistant to the spread of fire. Although wood will char where exposed to fire or high temperatures, even if it is treated with a fire retardant, chemicals will retard transmission of heat and rate of destruction. Treated with adequate quantities of an approved chemical, wood will not support combustion nor contribute fuel to a fire and will cease to burn after the ignition source is removed. The fire retardant may be applied as a paint or by impregnation under pressure. The latter is more effective. It may be considered permanent if the wood is used where it will be protected from the weather.

The effects of fire-retardant impregnation treatments on strength should be considered in design. Design values, including those for connections, for lumber and

structural glued-laminated timber pressure treated with fire-retardant chemicals should be obtained from the company providing the treatment and redrying service. Load duration factors greater than 1.6 (Table 10.5) should not be applied to structural members pressure-treated with fire-retardant chemicals.

10.7 DESIGN PROVISIONS FOR FLEXURAL MEMBERS

Design of flexural members requires consideration primarily of bending and shear strength, deflection, and end bearing.

10.7.1 Strength of Flexural Members

The stress induced in a beam (or other flexural member) when subjected to design loads should not exceed the strength of the member. The maximum bending stress f_b at any section of a beam is given by the flexural formula

$$f_b = M/S \quad (10.14)$$

where M is the bending moment and S the section modulus. For a rectangular beam, the section modulus is $bd^2/6$ and Eq. (10.14) transforms into

$$f_b = 6M/bd^2 \quad (10.15)$$

where b is the beam width and d the depth. At every section of the beam, f_b should be equal to or less than the design value for bending F_b adjusted for all end-use modification factors (Art. 10.5).

Shear stress induced by design loads in a member should not exceed the allowable design value for shear F_v . For wood beams, the shear parallel to the grain, that is, the horizontal shear, controls the design for shear. Checking the shear stress perpendicular to the grain is not necessary inasmuch as the vertical shear will never be a primary failure mode.

The maximum horizontal shear stress f_v in a rectangular wood beam is given by

$$f_v = 3V/2bd \quad (10.16)$$

where V is the vertical shear. In calculation of V for a beam, all loads occurring within a distance d from the supports may be ignored. This is based on the assumption that loads causing the shear will be transmitted at a 45° angle through the beam to the supports.

(K. F. Faherty and T. G. Williamson, "Wood Engineering and Construction Handbook." McGraw-Hill Publishing Company, New York.)

10.7.2 Beam Stability

Beams may require lateral support to prevent lateral buckling. Need for such bracing depends on the unsupported length and cross-sectional dimensions of the members. When buckling occurs, a member deflects in the direction of its least dimension b . In a beam, b usually is taken as the width. If bracing precludes buckling in that

direction, deflection can still occur, but in the direction of the strong dimension. Thus, the unsupported length L , width b , and depth d are key variables in formulas for lateral support and for reduction of design values for buckling.

Design for lateral stability of flexural members is based on a function of Ld/b^2 . The beam stability factor, C_L , for lumber beams of rectangular cross section having maximum depth-width ratios based on nominal dimensions, as summarized in Table 10.13, can be taken as unity.

No lateral support is required when the depth does not exceed the width of a beam. In that case also, the design value for bending does not have to be adjusted for lateral stability. Similarly, if continuous support prevents lateral movement of the compression flange, and the ends at points of bearing are braced to prevent rotation, then lateral buckling cannot occur and the design of value F_b need not be reduced.

When the beam depth exceeds the width, lateral support should be provided at end bearings. This support should be so placed as to prevent rotation of the beam about the longitudinal axis. Unless the compression flange is braced at sufficiently close intervals between supports, the design value should be adjusted for lateral buckling.

The slenderness ratio R_B for beams is defined by

$$R_B = \sqrt{\frac{L_e d}{b^2}} \quad (10.17)$$

The slenderness ratio should not exceed 50.

The beam stability factor C_L may be calculated from

$$C_L = \frac{1 + (F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1 + (F_{bE}/F_b^*)}{1.9} \right]^2 - \frac{F_{bE}/F_b^*}{0.95}} \quad (10.18)$$

TABLE 10.13 Approximate Lateral-Support Rules for Lumber Flexural Members*

Depth-width ratio (nominal dimensions)	Rule
$d/b \leq 2$	No lateral support required
$2 < d/b \leq 4$	Hold ends in position with full-depth blocking, bridging, hangers, or other structural members
$4 < d/b \leq 5$	Hold ends in position and compression edge in line, e.g., with direct connection of sheathing, decking, or joists
$5 < d/b \leq 6$	Hold ends in position and compression edge in line, as for 5 to 1, and provide adequate bridging or blocking at intervals not exceeding 8 ft
$6 < d/b \leq 7$	Hold ends in position and both edges firmly in line

If a beam is subject to both flexure and compression parallel to grain, the ratio may be as much as 5:1 if one edge is held firmly in line, e.g., by rafters (or roof joists) and diagonal sheathing. If the combined loads will induce tension on the unbraced face of the member, the ratio may be 6:1.

* From "National Design Specification for Wood Construction," American Forest & Paper Association.

where F_b^* = design value for bending multiplied by all applicable adjustment factors except C_{fu} , C_v , and C_L (Art. 10.5)

$$F_{bE} = K_{bE}E'/R_b^2$$

$$K_{bE} = 0.438 \text{ for visually graded lumber}$$

$$= 0.561 \text{ for machine evaluated lumber (MEL)}$$

$$= 0.609 \text{ for products with a coefficient of variation of 0.11 or less}$$

$$E' = \text{design modulus of elasticity multiplied by applicable adjustment factors (Art. 10.5)}$$

The effective length L_e for Eq. (10.17) is given in Table 10.14 in terms of the unsupported length of beam. Unsupported length is the distance between supports or the length of a cantilever when the beam is laterally braced at the supports to prevent rotation and adequate bracing is not installed elsewhere in the span. When both rotational and lateral displacement are also prevented at intermediate points, the unsupported length may be taken as the distance between points of lateral support. If the compression edge is supported throughout the length of the beam and adequate bracing is installed at the supports, the unsupported length is zero.

10.7.3 Deflection of Wood Beams

The design of many structural systems, particularly those with long span, may be governed by deflection. Verifying structural adequacy based on allowable stresses alone may not be sufficient to prevent excessive deflection. Limitations on deflection may increase member stiffness.

TABLE 10.14 Effective Length L_e for Lateral Stability of Beams*

Loading	$L_u/d \geq 7^\dagger$	$L_u/d < 7^\ddagger$
Simple beam§		
Uniformly distributed load	$1.63L_u + 3d$	$2.06L_u$
Load concentrated at midspan	$1.37L_u + 3d$	$1.80L_u$
Equal end moments	$1.84L_u$	$1.84L_u$
Equal concentrated loads at third points	$1.68L_u$	$1.68L_u$
Equal concentrated loads at quarter points	$1.54L_u$	$1.54L_u$
Equal concentrated loads at fifth points	$1.68L_u$	$1.68L_u$
Cantilever§		
Uniformly distributed load	$0.90L_u + 3d$	$1.33L_u$
Concentrated load on the end	$1.44L_u + 3d$	$1.87L_u$

* As specified in the "National Design Specification for Wood Construction," American Forest & Paper Association.

$^\dagger L_u$ = clear span when depth d exceeds width b and lateral support is provided to prevent rotational and lateral displacement at bearing points in a plane normal to the beam longitudinal axis and no lateral support is provided elsewhere.

$^\ddagger L_u$ = maximum spacing of secondary framing, such as purlins, when lateral support is provided at bearing points and the framing members prevent lateral displacement of the compression edge of the beam at the connections.

§ For a conservative value of L_e for any loading on simple beams or cantilevers, use $2.06 L_u$ when $L_u/d \leq 7$, and $1.63 L_u + 3d$ when $7 \leq L_u/d \leq 14.3$, and $1.84 L_u$ when $L_u/d \leq 14.3$.

Deflection of wood beams is calculated by conventional elastic theory. For example, for a uniformly loaded, simple-span beam, the maximum deflection is computed from

$$\Delta = 5wL^4/384EI \quad (10.19)$$

where w = the uniform load

L = span

E = modulus of elasticity

I = moment of inertia

Deflection should not exceed limitations specified in the local building code nor industry-recommended limitations. (See, for example, K. F. Faherty and T. G. Williamson, "Wood Engineering and Construction Handbook," McGraw-Hill Publishing Company, New York.) Deflections also should be evaluated with respect to other considerations, such as possibility of binding of doors or cracking of partitions or glass.

Table 10.15 gives recommended deflection limits, as a fraction of the beam span, for timber beams. The limitation applies to live load or total load, whichever governs.

Glulam beams may be cambered to offset the effects of deflections due to design loads. These members are cambered during fabrication by creation of curvature opposite in direction to that of deflections under load. Camber, however, does not increase stiffness. Table 10.16 lists recommended minimum cambers for glulam beams.

Minimum Roof Slopes. Flat roofs have collapsed during rainstorms even though they were adequately designed for allowable stresses and definite deflection limitations. The failures were caused by ponding of water as increasing deflections permitted more and more water to collect.

Roof beams should have a continuous upward slope equivalent to $\frac{1}{4}$ in/ft between a drain and the high point of a roof, in addition to minimum recommended

TABLE 10.15 Recommended Beam-Deflection Limitations, in
(In Terms of Span, l , in)

Use classification	Live load only	Dead load plus live load
Roof beams:		
Industrial	$l/180$	$l/120$
Commercial and industrial:		
Without plaster ceiling	$l/240$	$l/180$
With plaster ceiling	$l/360$	$l/240$
Floor beams:		
Ordinary usage*	$l/360$	$l/240$
Highway bridge stringers	$l/200$ to $l/300$	
Railway bridge stringers	$l/300$ to $l/400$	

* Ordinary usage classification is intended for construction in which walking comfort, minimized plaster cracking, and elimination of objectionable springiness are of prime importance. For special uses, such as beams supporting vibrating machinery or carrying moving loads, more severe limitations may be required.

TABLE 10.16 Recommended Minimum Camber for Glued-Laminated Timber Beams*

Roof beams†	1½ times dead-load deflection
Floor beams‡	1½ times dead-load deflection
Bridge beams:§	
Long span	2 times dead-load deflection
Short span	2 times dead-load plus ½ applied-load deflection

*Camber and Deflection, AITC 102, Appendix B, American Institute of Timber Construction.

†The minimum camber of 1½ times dead-load deflection will produce a nearly level member under dead load alone after plastic deformation has occurred. Additional camber is usually provided to improve appearance or provide necessary roof drainage.

‡The minimum camber of 1½ times dead-load deflection will produce a nearly level member under dead load alone after plastic deformation has occurred. On long spans, a level ceiling may not be desirable because of the optical illusion that the ceiling sags. For warehouse or similar floors where live load may remain for long periods, additional camber should be provided to give a level floor under the permanently applied load.

§Bridge members are normally cambered for dead load only on multiple spans to obtain acceptable riding qualities.

camber (Table 10.16), to avoid ponding. As a general guideline, when flat roofs have insufficient slope for drainage (less than ¼ in/ft), the stiffness of supporting members should be such that a 5-lb/ft² load will cause no more than ½-in deflection.

Because of ponding, snow loads or water trapped by gravel stops, parapet walls, or ice dams magnify stresses and deflections from existing roof loads by

$$C_p = \frac{1}{1 - W'L^3/\pi^4EI} \quad (10.20)$$

where C_p = factor for multiplying stresses and deflections under existing loads to determine stresses and deflections under existing loads plus ponding

W = weight of 1 in of water on roof area supported by beam, lb

L = span of beam, in

E = modulus of elasticity of beam material, psi

I = moment of inertia of beam, in⁴

(Kuenzi and Bohannon, "Increases in Deflection and Stresses Caused by Ponding of Water on Roofs," Forest Products Laboratory, Madison, Wis.)

10.7.4 Bearing Stresses in Beams

Bearing stresses, or compression stresses perpendicular to the grain, in a beam occur at the supports or at places where other framing members are supported on the beam. The compressive stress in the beam $f_{c\perp}$ is given by

$$f_{c\perp} = P/A \quad (10.21)$$

where P = load transmitted to or from the beam and A = bearing area. This stress should be less than the design value for compression perpendicular to the grain $F_{c\perp}$.

multiplied by applicable adjustment factors (Art. 10.5). (The duration-of-load factor does not apply to $F_{c\perp}$ for either solid sawn lumber or glulam timber.)

Limitations on compressive stress perpendicular to the grain are set to keep deformations within an acceptable range. An expected failure mode is excessive localized deformation rather than a catastrophic type of failure.

Design values for $F_{c\perp}$ are averages based on a maximum deformation of 0.04 in in tests conforming with ASTM D143. Design values $F_{c\perp}$ for glulam beams are generally lower than for solid sawn lumber with the same deformation limit. This is due partly to use of larger-size sections for glulam beams, length of bearing and partly to the method used to derive the design values.

Where deformations are critical, the deformation limit may be decreased, with resulting reduction in $F_{c\perp}$. For example, for a deformation maximum of 0.02 in. the "National Design Specification for Wood Construction," American Forest & Paper Association, recommends that $F_{c\perp}$, psi, be reduced to $0.73 F_{c\perp}$.

10.7.5 Example of Design of a Glulam Beam

Standard beam formulas for bending, shear, and deflection may be used to determine beam sizes. Ordinarily, bending deflection governs design; but for short, heavily loaded beams, shear may control.

Design values for bending are tabulated in the supplement to the "National Design Specification for Wood Construction." These values should be adjusted for service conditions (Art. 10.5). Section properties for solid sawn lumber and timber and glulam members are listed in the supplement to the "National Design Specification for Wood Construction," American Forest & Paper Association, Washington, D.C.

With the following data, design a straight glued-laminated roof beam, simply supported and uniformly loaded: span, 28 ft; spacing, 9 ft c to c; live load, such as snow, 30 lb/ft²; dead load, 5 lb/ft² for deck and 7.5 lb/ft² for roofing. Allowable design value for bending of glulam combination grade is 2400 psi, and for horizontal shear is 195 psi, for modulus of elasticity $E = 1,800,000$ psi. These are typical for a generic grade of glulam that can be manufactured using several different species. Deflection limitation for total load is $L/180$, where L is the span, ft. Assume the beam is laterally supported by the deck throughout its length.

With a 15% increase for load duration, such as snow, the allowable bending stress F_b becomes 2760 psi, and the allowable horizontal shear F_v , 224 psi.

Assume the beam will weigh 22.5 lb/lin ft, equivalent to 2.5 lb/ft² based on a 9-ft c to c spacing. Thus, the total uniform load comes to 45 psf. So the beam carries $w = 45 \times 9 = 405$ lb/lin ft.

The end shear $V = wL/2$ and the maximum shearing stress $= 3V/2 = 3wL/4$. Hence, the required area, in², for horizontal shear is

$$A = \frac{3wL}{4F_v} = \frac{wL}{299} = \frac{405 \times 28}{299} = 37.9 \text{ in}^2$$

The required section modulus, in³, is

$$S = \frac{1.5wL^2}{F_b} = \frac{1.5 \times 405 \times 28^2}{2760} = 172.6 \text{ in}^3$$

If $D = 180$, the reciprocal of the deflection limitation, then the deflection equals

$5 \times 1728wL^4/384EI \leq 12L/D$, where I is the moment of inertia of the beam cross section, in⁴. Hence, to control deflection, the moment of inertia must be at least

$$I = \frac{1.875DwL^3}{E} = \frac{1.875 \times 180 \times 405 \times 28^3}{1,800,000} = 1688 \text{ in}^4 \quad (10.22)$$

Assume that the beam will be fabricated with 1½-in laminations. From the table of section properties in the supplement to “National Design Specification for Wood Construction,” the most economical section satisfying all three criteria is 5⅝ × 16½, with $A = 84.6$, $S = 232.5$, and $I = 1919$. But it has a volume factor of 0.94%, assuming western species are used. So the allowable bending stress must be reduced to $2760 \times 0.94 = 2594$ psi. And the required section modulus must be increased accordingly to $172.6/0.94 = 183.6$. The selected section still is adequate.

10.7.6 Cantilevered-Span Construction

Cantilever systems may be composed of any of the various types and combinations of beam illustrated in Fig. 10.1. Cantilever systems permit longer spans or larger loads for a given size member than do simple-span systems, if member size is not controlled by compression perpendicular to grain at the supports or by horizontal shear. Substantial design economies can be effected by decreasing the depths of the members in the suspended portions of a cantilever system.

For economy, the negative bending moment at the supports of a cantilevered beam should be equal in magnitude to the positive moment.

Consideration must be given to deflection and camber in cantilevered multiple spans. When possible, roofs should be sloped the equivalent of ¼ in per foot of horizontal distance between the level of drains and the high point of the roof to eliminate water pockets, or provisions should be made to ensure that accumulation of water does not produce greater deflection and live loads than anticipated. Unbalanced loading conditions should be investigated for maximum bending moment, deflection, and stability.

(For further information on the design of cantilevered beam systems, see K. F. Faherty and T. G. Williamson, “Wood Engineering and Construction Handbook,” 2d ed., McGraw-Hill Publishing Company, New York; D. E. Breyer, “Design of Wood Structures,” 3d ed., McGraw-Hill Publishing Company, New York; “Wood Structural Design Data,” American Forest and Paper Association, Washington, D.C.)

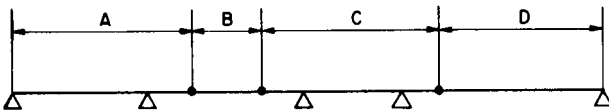


FIGURE 10.1 Cantilevered beam systems. *A* is a single cantilever, *B* is a suspended beam. *C* has a double cantilever, and *D* is a beam with the end suspended.

10.8 WOOD COMPRESSION MEMBERS

The design of wood columns or other types of compression members requires consideration of compressive strength parallel to the grain, end bearing, and stability, or resistance to buckling. Compressive strength considerations are the same regardless of the type of column, since the maximum compressive stress f_c induced by loads must not exceed the design value for compression parallel to the grain, F'_c , multiplied by applicable adjustment factors for service conditions (Art. 10.5). (For design for end bearing, see Art. 10.11.1, and for stability, see Art. 10.8.1).

Wood compression members may be a solid piece of lumber or timber (Fig. 10.2a), or spaced columns, connector joined (Fig. 10.2b and c), or built-up (Fig. 10.2d).

10.8.1 Solid Columns

These consist of a single piece of lumber or timber or of pieces glued together to act as a single member. In general,

$$f_c = P/A_g \leq F'_c \tag{10.23}$$

where P = axial load on the column

A_g = gross area of column

F'_c = design value in compression parallel to grain multiplied by the applicable adjustment factors, including column stability factor C_p

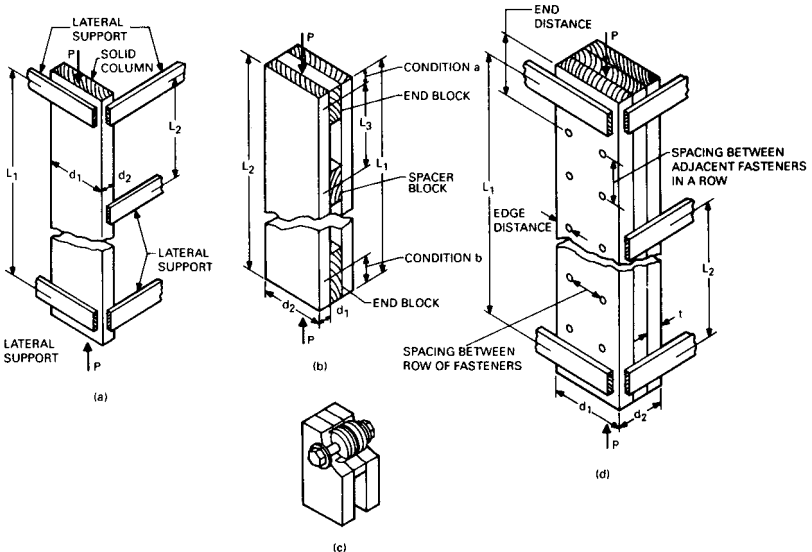


FIGURE 10.2 Behavior of wood columns depends on length-thickness or length-depth ratios: (a) solid wood column; (b) spaced column (the end distance for condition a should not exceed $L_1/20$ and for condition b should be between $L_1/20$ and $L_1/10$); (c) shear-plate connectors in the end-block of the spaced column; (d) built-up column.

There is an exception, however, applicable when holes or other reductions in area are present in the critical part of the column length most susceptible to buckling; for instance, in the portion between supports that is not laterally braced. In that case, f_c should be based on the net section and should not exceed F_c , the design value for compression parallel to grain, multiplied by applicable adjustment factors, except C_p ; that is,

$$f_c = P/A_n \leq F_c \quad (10.24)$$

where A_n = net cross-sectional area.

The stability factor represents the tendency of a column to buckle and is a function of the slenderness ratio. For a rectangular wood column, a modified slenderness ratio, L_e/d , is used, where L_e is the effective unbraced length of column, and d is the smallest dimension of the column cross section. The effective column length for a solid column should be determined in accordance with good engineering practice. The effective length L_e may be taken as the actual column length multiplied by the appropriate buckling-length coefficient K_e . For the solid column in Fig. 10.2a, the slenderness ratio should be taken as the larger of the ratios L_{e1}/d_1 or L_{e2}/d_2 , where each unbraced length is multiplied by the appropriate value of K_e . For solid columns, L_e/d should not exceed 50, except that during construction, L_e/d may be as large as 75.

The column stability factor C_p is given by

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c} \right]^2 - \frac{F_{cE}/F_c^*}{c}} \quad (10.25)$$

where F_c^* = design value for compression parallel to the grain multiplied by all applicable adjustment factors except C_p

$$F_{cE} = K_{cE}E'/(L_e/d)^2$$

E' = modulus of elasticity multiplied by adjustment factors

K_{cE} = 0.3 for visually graded lumber

= 0.384 for machine evaluated lumber (MEL)

= 0.418 for products with a coefficient of variation less than 0.11

c = 0.80 for solid sawn lumber

= 0.85 for round timber piles

= 0.90 for glulam timber

For a compression member braced in all directions throughout its length to prevent lateral displacement, $C_p = 1.0$.

10.8.2 Built-up Columns

These often are fabricated by joining together individual pieces of lumber with mechanical fasteners, such as nails, spikes, or bolts, to act as a single member (Fig. 10.2d). Strength and stiffness properties of a built-up column are less than those of a solid column with the same dimensions, end conditions, and material (equivalent solid column). Strength and stiffness properties of a built-up column, however, are much greater than those of an unconnected assembly in which individual pieces act as independent columns. Built-up columns obtain their efficiency from the increase in the buckling resistance of the individual laminations provided by the fasteners. The more nearly the laminations of a built-up column deform together—that is, the

smaller the slip between laminations, under compressive load—the greater is the relative capacity of the column compared with an equivalent solid column.

When built-up columns are nailed or bolted in accordance with provisions in the “National Design Specification for Wood Construction,” American Forest & Paper Association, the capacity of nailed columns exceeds 60% and of bolted built-up columns, 75% of an equivalent solid column for all L/d ratios. The NDS contains criteria for design of built-up columns based on tests performed on built-up columns with various fastener schedules.

10.8.3 Spaced Columns

A wood spaced column consists of the following elements: (1) two or more individual, rectangular wood compression members with their wide faces parallel; (2) wood blocks that separate the members at their ends and one or more points between; and (3) steel bolts through the blocks to fasten the components, with splitting or shear-plate connectors at the end blocks (Fig. 10.2*b*). The connectors should be capable of developing required shear resistance.

The advantage of a spaced column over an equivalent solid column is the increase permitted in the design value for buckling for the spaced-column members because of the partial end fixity of those members. The increased capacity may range from $2\frac{1}{2}$ to 3 times the capacity of a solid column. This advantage applies only to the direction perpendicular to the wide faces. Design of the individual members in the direction parallel to the wide faces is the same for each as for a solid column. The NDS gives design criteria, including end fixity coefficients, for spaced columns.

10.9 TENSION MEMBERS

The tensile stress f_t , parallel to the grain should be computed for the net section area. This stress should not exceed the design value for tension parallel to grain F_t .

Designs that induce tensile stress perpendicular to the grain should be avoided. The reason for this is that wood is weaker and more variable in tension perpendicular to the grain than in other properties. Furthermore, these tensile properties have not been extensively evaluated and published values are not readily available. When tension perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist the stresses may be required. An example of a construction that induces critical tensile stress perpendicular to grain is a load supported from a beam from a point below the neutral axis. This practice should be avoided for medium to heavy loads.

10.10 COMBINED BENDING AND AXIAL LOADING

When a bending moment and an axial force act on a section of a structural member, the effects of the combined stresses must be provided for in design of the member.

10.10.1 Bending and Axial Tension

Members subjected to combined bending and axial tension should be proportioned to satisfy the interaction equations, Eqs. (10.26) and (10.27).

$$\frac{f_t}{F'_t} + \frac{f_b}{F_b^*} \leq 1 \quad (10.26)$$

$$(f_b - f_t) / F_b^{**} \leq 1 \quad (10.27)$$

where f_t = tensile stress due to axial tension acting alone

f_b = bending stress due to bending moment alone

F'_t = design value for tension multiplied by applicable adjustment factors

F_b^* = design value for bending multiplied by applicable adjustment factors except C_L

F_b^{**} = design value for bending multiplied by applicable adjustment factors except C_V

Adjustment factors are discussed in Art. 10.5.

The load duration factor C_D associated with the load of shortest duration in a combination of loads with differing duration may be used to calculate F'_t and F_b^* . All applicable load combinations should be evaluated to determine the critical load combination.

10.10.2 Bending and Axial Compression

Members subjected to a combination of bending and axial compression (beam-columns) should be proportioned to satisfy the interaction equation, Eq. 10.28.

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b1}}{[1 - (f_c/F_{cE1})] F'_{b1}} + \frac{f_{b2}}{[1 - (f_c/F_{cE2}) - (f_{b1}/F'_{b1})^2] F'_{b2}} \leq 1 \quad (10.28)$$

where f_c = compressive stress due to axial compression acting alone

F'_c = design value for compression parallel to grain multiplied by applicable adjustment factors, including the column stability factor

f_{b1} = bending stress for load applied to the narrow face of the member

f_{b2} = bending stress for load applied to the wide face of the member

F'_{b1} = design value for bending for load applied to the narrow face of the member multiplied by applicable adjustment factors, including the beam stability factor

F'_{b2} = design value for bending for load applied to the wide face of the member multiplied by applicable adjustment factors, including the beam stability factor

For either uniaxial or biaxial bending, f_c should not exceed

$$F_{cE1} = K_{cE} E' / (L_{e1} / d_1)^2 \quad (10.29a)$$

Also, for biaxial bending, f_c should not exceed

$$F_{cE2} = K_{cE} E' / (L_{e2} / d_2)^2 \quad (10.29b)$$

and f_{b1} should not be more than

$$F_{bE} = K_{bE}E'/R_B^2 \quad (10.30)$$

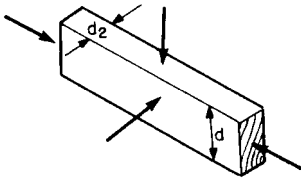


FIGURE 10.3 Beam-column.

where d_1 = width of the wide face (Fig. 10.3) and d_2 = width of the narrow face (Fig. 10.3). Slenderness ratio R_B for beams is given by Eq. (10.17). K_{bE} is defined for Eq. (10.18). The effective column lengths L_{e1} for buckling in the d_1 direction and L_{e2} for buckling in the d_2 direction, E' , F'_{cE1} , and F'_{cE2} should be determined in accordance with Art. 10.8.1. Adjustment factors are discussed

in Art. 10.5. The load duration factor C_D should be applied in calculation of F'_c , and F'_{b1} , and F'_{b2} as indicated for combined bending and axial tension.

10.11 BEARING STRESSES

These may occur in a wood structural member parallel to the grain (end bearing), perpendicular to the grain, or at an angle to the grain.

10.11.1 Bearing Parallel to Grain

The bearing stress parallel to grain f_g should be computed for the net bearing area. This stress may not exceed the design value for bearing parallel to grain F_g multiplied by load duration factor C_D and temperature factor C_t (Art. 10.5). The adjusted design value applies to end-to-end bearing of compression members if they have adequate lateral support and their end cuts are accurately squared and parallel to each other.

When f_g exceeds 75% of the adjusted design value, the member should bear on a metal plate, strap, or other durable, rigid, homogeneous material with adequate strength. In such cases, when a rigid insert is required, it should be a steel plate with a thickness of 20 ga or more or the equivalent thereof, and it should be inserted with a snug fit between abutting ends.

10.11.2 Bearing Perpendicular to Grain

This is equivalent to compression perpendicular to grain. The compressive stress should not exceed the design value perpendicular to grain multiplied by applicable adjustment factors, including the bearing area factor (Art. 10.5.11). In the calculation of bearing area at the end of a beam, an allowance need not be made for the fact that, as the beam bends, it creates a pressure on the inner edge of the bearing that is greater than at the end of the beam.

10.11.3 Bearing at an Angle to Grain

The design value F_g for bearing parallel to grain and the design value for bearing perpendicular to grain $F_{c\perp}$ differ considerably. When load is applied at an angle θ

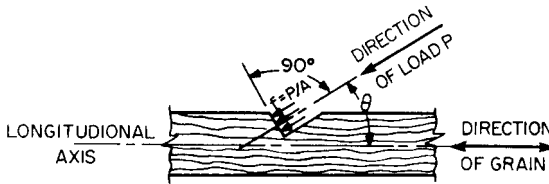


FIGURE 10.4 Load applied to a wood member at an angle to the grain.

with respect to the grain, where $0 \leq \theta \leq 90^\circ$ (Fig. 10.4), the design value for bearing lies between F'_g and $F'_{c\perp}$. The “National Design Specification for Wood Construction,” American Forest & Paper Association, recommends that the design value for such loading be calculated from the Hankinson formula:

$$F'_\theta = \frac{F'_g F'_{c\perp}}{F'_g \sin^2 \theta + F'_{c\perp} \cos^2 \theta} \quad (10.31)$$

where F'_θ = adjusted design value for bearing at angle θ to the grain (longitudinal axis)

F'_g = design value for end bearing multiplied by applicable adjustment factors

$F'_{c\perp}$ = design value for compression perpendicular to grain multiplied by applicable adjustment factors

10.12 STRUCTURAL PANELS

Wood-based structural panels are thin, flat, composite materials capable of resisting applied loads in specific applications. Structural panels fall into three basic categories based on the manufacturing process used: plywood, mat-formed panels (oriented strand board, or OSB), and composite panels.

Plywood—a flat panel built up of sheets of veneer, called plies. These are united under pressure by a bonding agent. The adhesive bond between plies is as strong as or stronger than solid wood. Plywood is constructed of an odd number of layers with the grain of adjacent layers perpendicular. Layers may consist of a single ply or two or more plies laminated with parallel grain direction. Outer layers and all odd-numbered layers generally have the grain direction oriented parallel to the long dimension of the panel. The odd number of layers with alternating grain direction equalizes strains, reduces splitting, and minimizes dimensional change and warping of the panel.

Mat-formed panel—any wood-based panel that does not contain veneer but is consistent with the definition of structural-use panels, including products such as waferboard and oriented strand board.

Oriented strand board—an engineered structural wood panel composed of compressed wood strands arranged in layers at right angles to one another and bonded with fully waterproof adhesive.

Composite panel—any panel containing a combination of veneer and other wood-based materials.

A structural panel may contain either softwoods or hardwoods. Panels approved for use in building-code-regulated construction carry the trademark of a code-approved agency, such as APA—The Engineered Wood Association. Most construction grades have either an Exterior or Exposure 1 durability classification and are made with fully waterproof adhesives. Exposure classifications are defined as follows:

Exterior—panels that are suitable for permanent exposure to weather or moisture.

Exposure 1—panels that are suitable for uses not permanently exposed to the weather but may be used where exposure durability to resist effects of moisture due to construction delays, high humidity, water leakage, or other conditions of similar severity is required.

Exposure 2—panels that are suitable for interior use where exposure durability to resist effects of high humidity and water leakage is required.

Interior—panels that are suitable for interior use where they will be subjected to only temporary, minor amounts of moisture.

10.12.1 Standards for Structural Panels

Structural panels approved for building-code-regulated construction are manufactured under one or more of three standards:

1. *U.S. Product Standard PS 1-95 for Construction & Industrial Plywood (PS 1)*. It applies to plywood only. This voluntary product standard covers the wood species, veneer grading, glue bonds, panel construction and workmanship, dimensions and tolerances, marking, moisture content, and packing of plywood intended for construction and industrial uses. Also included are test methods to determine compliance and a glossary of trade terms and definitions. A quality certification program is provided, whereby qualified testing agencies inspect, sample, and test products identified as complying with the standard. Information regarding generally available sizes, methods of ordering, and reinspecting practices also is provided.

2. *Voluntary Product Standard PS 2-92, Performance Standard for Wood-Based Structural-Use Panels (PS 2)*. It applies to all types of wood-based panels (typically plywood, OSB, and composite). It establishes requirements for assessing the acceptability of wood-based structural-use panels for construction sheathing and single-floor applications. It also provides a basis for common understanding among the producers, distributors, and users of these products. It covers performance requirements, adhesive bond durability, panel construction and workmanship, dimensions and tolerances, marking, and moisture content of structural-use panels. The standard also includes test methods to determine compliance and a glossary of trade terms and definitions. A quality certification program is provided, whereby qualified testing agencies inspect, sample, and test products for qualification under the standard.

3. *APA Performance Standards and Policies for Structural-Use Panels (PRP 108)*. It is similar to PS 2 but also includes performance-based qualification procedures for siding panels.

10.12.2 Plywood Grades

Plywood grades are generally identified in terms of the veneer grade used on the face and back of the panel; for example, A-B, B-C, . . . , or by a name suggesting

the panel's intended end use, such as APA Rated Sheathing or APA Rated Sturd-I-Floor.

Veneer grades define veneer appearance in terms of natural, unrepaid-growth characteristics and allowable number and size of repairs that may be made during manufacture (Table 10.17). The highest quality veneer grades are N and A. The minimum grade of veneer permitted in Exterior plywood is C grade. D-grade veneer is used in panels intended for interior use or applications protected from permanent exposure to weather.

Panels with B-grade or better veneer faces are always sanded smooth in the manufacturing process to fulfill the requirements of their intended end use—applications such as cabinets, shelving, furniture, and built-ins. Rated Sheathing panels are unsanded since a smooth surface is not a requirement of their intended end use. Still other panels, such as Underlayment, Rated Sturd-I-Floor, C-D Plugged, and C-C Plugged, require only touch sanding for “sizing” to make the panel thickness more uniform.

Unsanded and touch-sanded panels, and panels with B-grade or better veneer on one side only, usually carry the trademark on the panel back. Panels with both sides of B-grade or better veneer, or with special overlaid surfaces, such as High-Density Overlay, usually carry the trademark on the panel edge.

10.12.3 Plywood Group Number

Plywood can be manufactured from over 70 species of wood. These species are divided on the basis of strength and stiffness into five groups under U.S. Product Standard PS 1-95. Strongest species are in Group 1; the next strongest in Group 2, etc. The group number that appears in the trademark on some APA trademarked

TABLE 10.17 Veneer-Grade Designations

Grade	Description
N	Smooth surface “natural finish” veneer. Select, all heartwood or all sapwood. Free of open defects. Allows not more than six repairs, wood only, per 4- × 8-ft panel, made parallel to grain and well matched for grain and color.
A	Smooth, paintable. Not more than 18 neatly made repairs, boat, sled, or router type, and parallel to grain, permitted. May be used for natural finish in less-demanding applications. Synthetic repairs permitted.
B	Solid surface. Shims, circular repair plugs, and tight knots up to 1 in across grain permitted. Some minor splits and synthetic repairs permitted.
C Plugged	Improved C veneer with splits limited to 1/8-in width and knotholes and borer holes limited to 1/4 × 1/2 in. Admits some broken grain. Synthetic repairs permitted.
C	Tight knots to 1 1/2 in. Knotholes up to 1 in across grain and some up to 1 1/2 in if total width of knots and knotholes is within specified limits. Synthetic or wood repairs and discoloration and sanding defects that do not impair strength permitted. Limited splits allowed. Stitching permitted.
D	Knots and knotholes up to 2 1/2-in wide across grain and 1/2 in larger within specified limits, limited splits, and stitching permitted. Limited to Exposure 1 or interior panels.

panels, primarily sanded grades, is based on the species used for face and back veneers. Where face and back veneers are not from the same species group, the higher group number is used, except for sanded panels $\frac{3}{8}$ in thick or less and decorative panels of any thickness. These are identified by face species if C or D grade backs are at least $\frac{1}{8}$ in thick and are not more than one species group number larger. Some species are used widely in plywood manufacture, others rarely.

OSB panels, being composed of flakes or strands instead of veneers, are graded without reference to veneers or species, and composite panels are graded on an OSB performance basis by end use and exposure durability. Typical panel trademarks for all three panel types and an explanation of how to read them are shown in Fig. 10.5.

The “Design/Construction Guide—Residential & Commercial,” APA—The Engineered Wood Association, Tacoma, Wash., contains a comprehensive summary of plywood grades and trademarks and their applications.

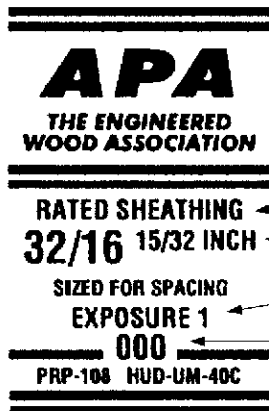
10.12.4 Span Ratings for Panels

APA Rated Sheathing, APA Rated Sturd-I-Floor, and APA Rated Siding carry numbers in their trademarks called span ratings. These denote the maximum spacing, in, c to c of supports for panels in construction applications. Except for Rated Siding panels, the span rating in the trademark applies when the long panel dimension is across supports, unless the strength axis is otherwise identified. The span rating in the trademark of Rated Siding panels applies when they are installed vertically.

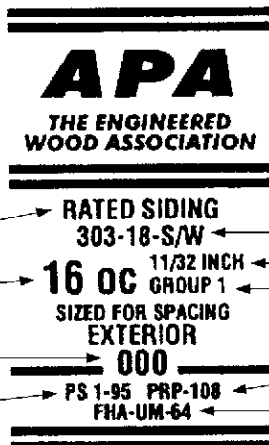
The span rating in Rated Sheathing trademarks appears as two numbers separated by a slash (Fig. 10.5a), such as 32/16 and 48/24. The left-hand number denotes the maximum recommended spacing of supports when the panel is used for roof sheathing with the long dimension or strength axis of the panel across three or more supports. The right-hand number indicates the maximum recommended spacing of supports when the panel is used for subflooring with the long dimension or strength axis of the panel across three or more supports. A panel marked 32/16, for example, may be used for roof decking over supports 32 in c to c or for subflooring over supports 16 in c to c. An exception is Rated Sheathing intended for use as wall sheathing only. The trademarks for such panels contain only a single number similar to the span rating for APA Rating Siding and Sturd-I-Floor.

The Span Ratings in the trademarks on APA Rated Sturd-I-Floor and APA Rating Siding panels appear as a single number. Rated Sturd-I-Floor panels are designed specifically for single-floor (combined subfloor-underlayment) applications under carpet and pad. They are manufactured with span ratings of 16, 20, 24, 32, and 48 in.

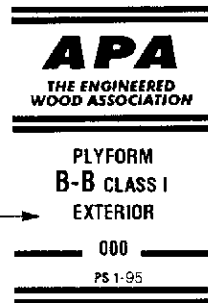
APA Rated Siding is available with span ratings of 16 and 24 in. Span-rated panels and lap siding may be connected directly to studs, or over nonstructural wall sheathing, or over nailable panel or lumber sheathing (double-wall construction). Panels and lap siding with a span rating of 16 in may be applied directly to studs spaced 16 in c to c. Those bearing a span rating of 24 in may be connected directly to studs 24 in c to c. All APA Rated Siding panels may be applied horizontally directly to studs 16 or 24 in c to c, if horizontal joints are blocked. The span rating of APA Rated Siding panels refers to the maximum recommended spacing of vertical rows of nails rather than to stud spacing when the panels are applied to nailable structural sheathing.



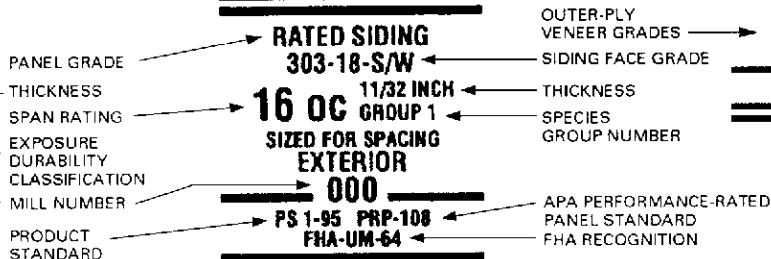
(a)



(b)



(c)



HDO • A-A • EXT-APA • 000 • PS1-95

(d)

MARINE • A-A • EXT-APA • 000 • PS1-95

(e)

FIGURE 10.5 Typical trademarks for structural panels. (a) APA Rated Sheathing with a thickness of $1\frac{1}{32}$ in and a span rating of 32 in for use as roof decking and 16 in for use as subflooring, suitable for Exposure 1 conditions (not permanently exposed to weather). (b) APA Rated Siding, grade 303-18-S/W, with a span rating of 16 in. (c) APA Plyform, intended for use in formwork for concrete. (d) APA high-density overlay (HDO), abrasion resistant and suitable for exterior applications (used for concrete forms, cabinets, countertops, and signs). (e) APA Marine, used for boat hulls.

10.12.5 Availability of Panel Grades

Some panel grades, thicknesses, span ratings, or species may be difficult to obtain in some areas. Check with your supplier for availability or include an alternative panel in specifications. Standard panel dimensions are 4 × 8 ft, although some mills also produce plywood panels 9 or 10 ft long or longer. OSB panels may be ordered in lengths up to 24 ft in some market areas.

10.12.6 APA Rated Sturd-I-Floor

APA Rated Sturd-I-Floor (copyrighted name) is a span-rated product designed specifically for use in single-layer floor construction beneath carpet and pad. The maximum spacing of floor joists, or span rating, is stamped on each panel. Panels are manufactured with span ratings of 16, 20, 24, 32, and 48 in. These assume the panel continuous over two or more spans with the long dimension or strength axis across supports (Fig. 10.6). The span rating in the trademark applies when the long panel dimension is across supports unless the strength axis is otherwise identified.

Glue-nailing is recommended, though panels may be nailed only. Application provisions for both methods are given in Table 10.18. Uniform live loads are given in the APA “Design/Construction Guide—Residential & Commercial.”

10.12.7 Panel Subflooring

The limiting factor in design of floors is deflection under concentrated loads at panel edges. Nailing provisions for APA panel subflooring (Fig. 10.7) are given in Table 10.19. Other code-approved fasteners, however, may be used. The span ratings in Table 10.19 applied to Rated Sheathing or sheathing grades only and are the minimum for the span indicated. The span ratings assume panels continuous over two or more spans with the long dimension or strength axis across supports.

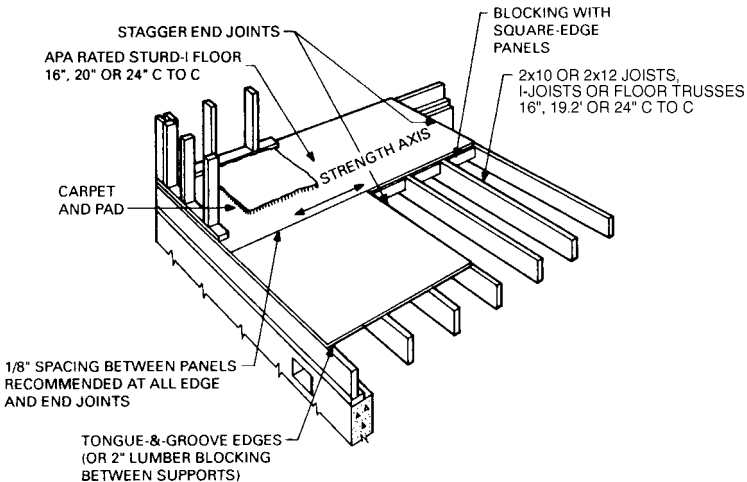


FIGURE 10.6 Floor construction with APA Rated Sturd-I-Floor.

TABLE 10.18 Fastener Size, Type, and Spacing for APA Rated Sturd-I-Floor^a

Span rating (maximum joist spacing), in	Minimum panel thickness, in ^b	Glue Nailed ^c			Nailed Only		
		Nail size and type	Spacing, in		Nail size and type	Spacing, in	
			Supported panel edges	Intermediate supports		Supported panel edges	Intermediate supports ^d
16	$\frac{19}{32}$ $\frac{5}{8}$	6d ring- or screw-shank ^e	12	12	6d ring- or screw-shank	6	12
20	$\frac{19}{32}$ $\frac{5}{8}$	6d ring- or screw-shank ^e	12	12	6d ring- or screw-shank	6	12
24	$\frac{23}{32}$ $\frac{3}{4}$	6d ring- or screw-shank ^e	12	12	6d ring- or screw-shank	6	12
	$\frac{7}{8}$	8d ring- or screw-shank ^e	6	12	8d ring- or screw-shank	6	12
32	$\frac{7}{8}$	8d ring- or screw-shank ^e	6	12	8d ring- or screw-shank	6	12
48	$\frac{3}{32}$ $\frac{1}{8}$	8d ring- or screw-shank ^f	6	^g	8d ring- or screw-shank	6	^g

^aHeavy traffic and concentrated loads may require construction in excess of the minimum values in the table.

^bPanels of a specific thickness may have more than one span rating. Panels with a span rating larger than the joist spacing may be substituted for panels of the same thickness with a span rating equal to the joist spacing.

^cAdhesives should conform to APA Specification AFG-01. Only solvent-based glues should be used for nonveneer panels with sealed surfaces and edges.

^dLocal building code may require nail spacing 10 in c to c at intermediate supports for floors.

^e8d common nails may be used if these nails are not available.

^f10d common nails may be used with $\frac{1}{8}$ -in panels if supports are well seasoned.

^gNails should be spaced 6 in c to c for 48-in spans and 12 in c to c for 32-in spans.

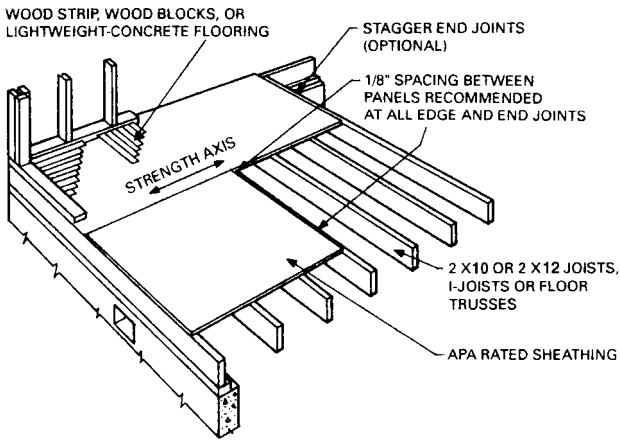


FIGURE 10.7 Subfloor constructed of structural panels.

TABLE 10.19 Spans and Nailing for APA Panel Subflooring^a
(APA Rated Sheathing)

Span rating or group no.	Minimum panel thickness, in	Maximum span, in	Nail size and type ^b	Nail spacing, in	
				At supported edges	At intermediate supports ^c
24/16	$\frac{7}{16}$	16	6d common	6	12
32/16	$\frac{15}{32}, \frac{1}{2}$	16 ^d	8d common ^e	6	12
40/20	$\frac{19}{32}, \frac{5}{8}$	20 ^{d,f}	8d common	6	12
48/24	$\frac{23}{32}, \frac{3}{4}$	24	8d common	6	12

^aFor recommendations for subfloors under ceramic tile, see the APA "Design/Construction Guide—Residential and Construction." For subfloors under gypsum concrete, obtain data from topping producers.

^bOther code-approved fasteners may be used

^cLocal building codes may require nail spacing 10 c to c at intermediate supports for floors.

^dA 24-in span may be used if $\frac{3}{4}$ -in-thick wood strip flooring is installed perpendicular to joists.

^eIf the panel is $\frac{1}{2}$ in or less thick, 6d common nails are permitted.

^fA 24-in span may be used if at least $\frac{1}{2}$ in of lightweight concrete is applied over the panels.

Panel subflooring may also be glued for added stiffness and to reduce squeaks if it satisfied nailing provisions in Table 10.18. Long edges should be tongue-and-groove or supported with blocking unless:

1. A separate underlayment layer is installed with its joints offset from those in the subfloor. The minimum thickness of underlayment should be $\frac{1}{4}$ in for subfloors on spans up to 24 in and $\frac{11}{32}$ in or more on spans longer than 24 in.
2. A minimum of $\frac{1}{2}$ in of lightweight concrete is applied over the panels.
3. A $\frac{3}{4}$ -in wood strip flooring is installed over the subfloor.

In some nonresidential buildings, greater traffic and heavier concentrated loads may require construction in excess of the minimums given. Where joists are 16 in

c to c, for example, panels with a span rating of 40/20 or 48/24 provide greater stiffness. For beams or joists 24 or 32 in c to c, 1 $\frac{1}{8}$ -in-thick panels provide additional stiffness.

10.12.8 Wall Systems

Rated siding (panel or lap) may be applied directly to studs or over nonstructural fiberboard, or gypsum or rigid-foam-insulation sheathing. Nonstructural sheathing is defined as sheathing not recognized by building codes as meeting both bending and racking-strength requirements.

A single layer of panel siding, since it is strong and rack resistant, eliminates the cost of installing separate structural sheathing or diagonal wall bracing. Panel sidings are normally installed vertically (Fig. 10.8a), but most may also be placed horizontally (long dimension across supports) if horizontal joints are blocked (Fig. 10.8b). Maximum stud spacings for both applications are given in Table 10.20.

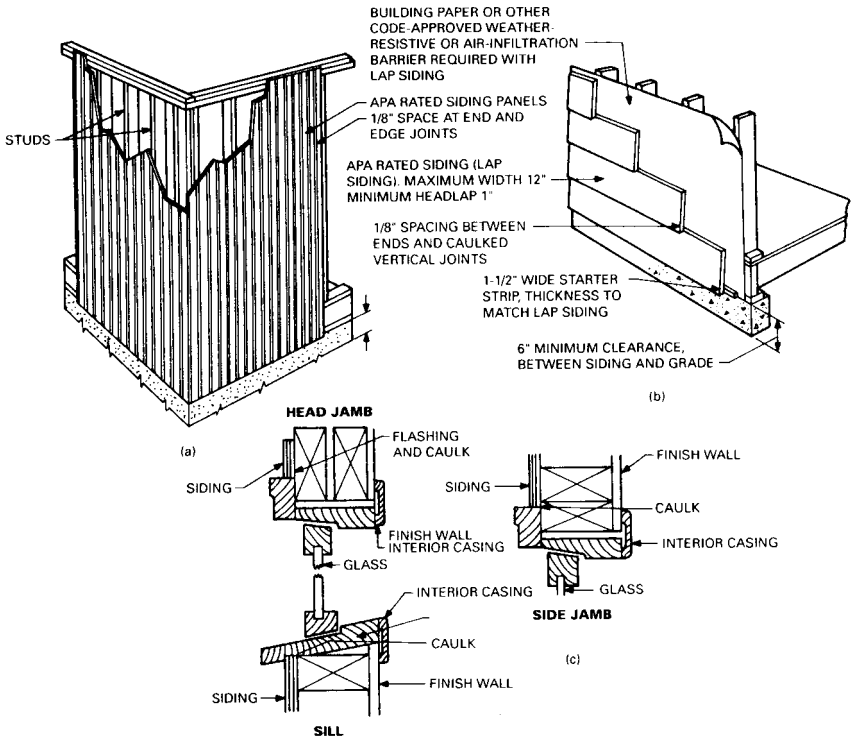


FIGURE 10.8 Wall built of wood studs and APA Rated Siding panels. (a) Vertical panel siding. If permitted by the local building and energy codes, no building paper is required when panel edges are shiplapped, battened, and caulked. If caulking is not used with unbattened square butt joints, apply a water repellent to panel edges. Caulk around windows and doors. (b) Horizontal lap siding. Diagonal bracing or other code-approved bracing methods for the wall should be provided. For engineered shear-wall segments, use APA Rated Sheathing under the lap siding. (c) Siding joint details at a window.

TABLE 10.20 Recommended Installation of APA Rated Sturd-I-Wall

	Siding Description ^a	Nominal thickness, in, or span rating	Maximum stud spacing, in		Nail size (nonstaining box, siding or casing nails) ^{b,c}	Nail spacing, ^d in	
			Long dimension vertical	Long dimension horizontal		At panel edges	At intermediate supports
Panel siding	APA MDO Exterior	¹ / ₃₂ and ³ / ₈	16	24	6d for siding ¹ / ₂ in thick or less; 8d for thicker siding.	6 ^e	12 ^f
		¹⁵ / ₃₂ and thicker	24	24			
	APA Rated Siding Exterior	16	16	16 ^e			
		24	24	24			
Lap siding	APA Rating Siding—Lap Exterior	16		16	6d for siding ¹ / ₂ in thick or less; 8d for thicker siding.	16 along bottom edge	
		24		24		24 along bottom edge	

^aRecommendations apply to all species groups for veneered APA Rated Siding, including APA 303 siding.

^bNext regular size nailing should be used if panel siding is applied over foam-insulation sheathing. If lap siding is installed over such sheathing up to 1 in thick, 10d (3-in) nails should be used for ³/₈- or ⁷/₁₆-in siding, 12d (3 ¹/₄-in) nails for ¹⁵/₃₂- or ¹/₂-siding, and 16d (3 ¹/₂-in) nails for ¹⁹/₃₂-in or thicker siding. Nonstaining box nails should be used for siding installed over foam-insulation sheathing.

^cHot-dipped or hot-tumbled galvanized steel nails are recommended for most siding applications, but electrically or mechanically galvanized nails are acceptable if the plating meets or exceeds thickness requirements of ASTM A641 Class 2 coatings and is protected by a yellow chromate coating. For best performance, stainless steel or aluminum nails are an alternative. Galvanized nails may react adversely under wet conditions with some wood species and cause staining if left unfinished. Such staining can be minimized if the siding is finished in accordance with APA recommendations or if the siding is protected by a roof overhang from direct exposure to moisture and weathering.

^dRecommendations of siding manufacturers may vary.

^eNails should be spaced 3 in c to c along panel edges for a braced wall section with ¹/₃₂- or ³/₈-in panel siding applied horizontally over studs spaced 24 in c to c.

^fWhere wind velocity exceeds 80 mph, nails attaching siding to intermediate studs within 10% of the width of the narrow side from wall corners should be spaced 6 in c to c.

^gStud spacing may be 24 in for veneer-faced siding panels.

Rated Sheathing meets building-code wall-sheathing requirements for bending and racking strength without let-in corner bracing. Installation provisions are given in Table 10.21 and Fig. 10.9. When 1/2-in gypsum or fiberboard sheathing is used, APA Rated Sheathing corner panels of the same thickness can also eliminate costly let-in bracing. The APA Rated Sheathing, $1\frac{5}{32}$ or 1/2 in thick, should be nailed to studs spaced 16 or 24 in c to c with 6d common nails spaced 6 in c to c along panel edges and 12 in c to c at intermediate supports. When corner panels are PS 1 plywood, 1/2-in roofing nails at 4 in along panel edges and 12 in at intermediate supports may be used.

TABLE 10.21 Recommended Installation of APA Panel Wall Sheathing^a

Panel span rating	Maximum stud spacing, in	Nail size ^b	Nail spacing, in	
			At supported panel edges	At intermediate supports
12/0, 16/0, 20/0 or Wall-16 in c to c	16	6d panels 1/2 in thick or less; 8d for thicker panels	6	12
24/0, 24/16, 32/16 or Wall-24 in c to c	24			

^aApplies to APA Rated Sheathing panels that are continuous over two or more spans. Different requirements may apply to nailable panel sheathing when the exterior covering is to be nailed to the sheathing. See the APA Design/Guide.

^bFor common, smooth, annular, spiral-thread, or galvanized box nails. Other code approved fasteners, however, may be used.

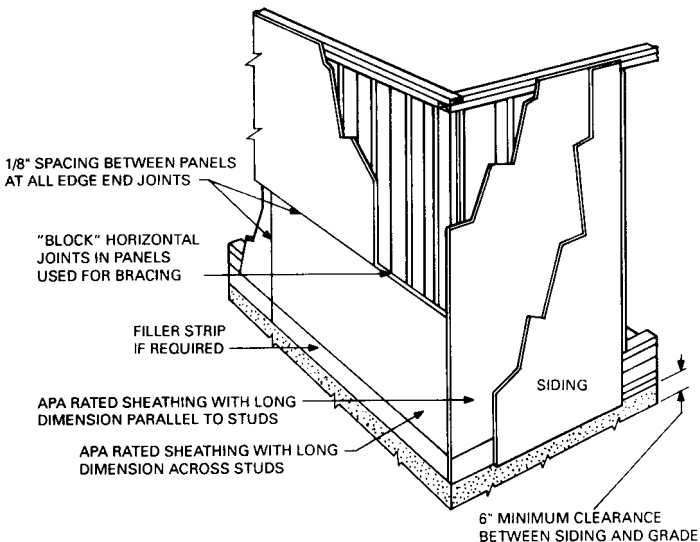


FIGURE 10.9 Application of structural panels as sheathing. Building paper is not required over the sheathing, except under stucco and brick veneer.

Building paper is generally not required over wall sheathing, except under stucco or under brick veneer where required by the local building code. Recommended wall sheathing spans with brick veneer and masonry are the same as those for nailable panel sheathing.

10.12.9 Allowable Loads for APA Structural-Use Panels

Because it is sometimes necessary to have engineering design information for structural panel products for conditions not specifically covered in other literature, APA publishes separate design-section capacities for the various span ratings for these products. These values are listed in APA Technical Note N375, "Design Capacities of APA Performance-Rated Structural-Use Panels." The APA "Plywood Design Specification" contains load-span tables that apply to APA trademarked structural-use panels qualified and manufactured in accordance with PS 2-92 or APA PRP-108, "Performance Standards and Policies for Structural-Use Panels." These panels include plywood, composite, and mat-formed products, such as oriented strand board. Loads are provided for applications where the panel strength axis is applied across or parallel to supports. For each combination of span L and Span Rating, loads are given for deflections of $L/360$, $L/240$ and $L/180$, and maximum loads controlled by bending and shear capacity. The values may be adjusted for panel type, load duration, span conditions, and moisture. Table 10.22 is provided to assist in selecting panel constructions for specific span ratings.

Some structural-panel applications are not controlled by uniform loads. Residential floors are a good example. They are commonly designed for 40-psf live load. The allowable uniform floor load on panels with maximum span according to APA recommendations is greatly in excess of typical design loads. This excess does not mean that floor spans for structural panels can be increased, but only that there is considerable reserve strength and stiffness for *uniform* loads. Actually, the

TABLE 10.22 Typical APA Panel Considerations*

Span rating	Plywood			COM-PLY	OSB
	3-Ply	4-Ply	5-Ply†		
APA Rated Sheathing					
24/0	X				X
24/16					X
32/16	X	X	X		X
40/20	X	X	X		X
48/24		X	X		X
APA Rated Sturd-I-Floor					
16 in c to c					
20 in c to c		X	X	X	X
24 in c to c		X	X	X	X
32 in c to c			X	X	X
48 in c to c			X	X	X

* Constructions may not be available in every area. Check with suppliers concerning availability.

† Applies to plywood with five or more layers.

recommendations for panel floors are based on performance under concentrated loads, how the floor “feels” to passing foot traffic, and other subjective factors that relate to public acceptance. The maximum floor and roof spans for structural panels should always be checked before a final panel selection is made for these applications.

10.12.10 Panel Shear Walls

While the wall systems described in Art. 10.12.8 will provide sufficient strength under normal conditions in residential and light-frame construction, shear walls may be desirable or required in areas with frequent seismic activity or high wind loads. Shear walls are also advisable in commercial and industrial construction.

Either Rated Sheathing or all-veneer plywood Rated Siding can be used in shear walls. Table 10.23 gives maximum shears for walls with Rated Sheathing, with plywood Rated Siding installed directly to studs (Sturd-I-Wall), or with panels applied over gypsum sheathing for walls required to be fired rated from the outside.

To design a shear wall, follow these steps:

1. Determine lateral loads and resulting shears with appropriate allowances for openings.
2. Determine the required panel grade and thickness and the nailing schedule from Table 10.23. Check the anchor bolts in the sill plate for shear.
3. Check wall framing on each end of the shear wall and design a foundation anchor or hold-down, if required (see Fig. 10.10).

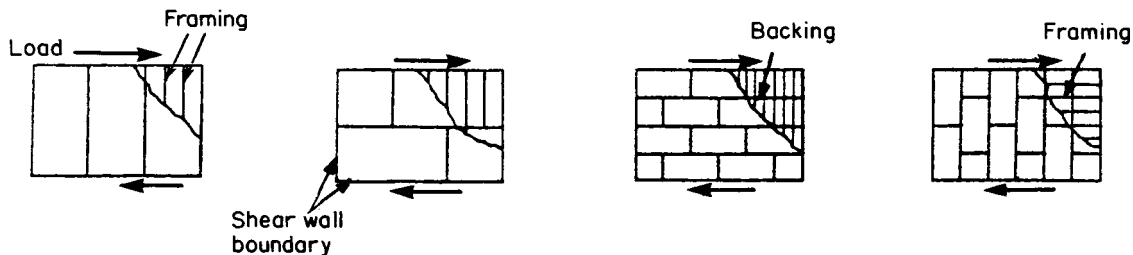
10.12.11 Panel Roof Sheathing

Table 10.24 lists maximum uniform roof live loads for APA Rated Sheathing Exposure 1, and Structural I Rated Sheathing, Exposure 1 or Exterior. Uniform-load deflection limits are $\frac{1}{180}$ of the span under live load plus dead load, and $\frac{1}{240}$ under live load only. Panels are assumed continuous over two or more spans with the long dimension or strength axis across supports (Fig. 10.11). Special conditions, such as heavy concentrated loads, may require constructions in excess of these minimums, or allowable live loads may have to be decreased for dead loads greater than 10 psf, for example, for tile roofs.

Good performance of built-up, single-ply, or modified bitumen roofing applied on low-slope roofs requires a stiffer deck than does prepared roofing applied on pitched roofs. Although Span-Rated panels used as roof sheathing at maximum span are adequate structurally, an upgraded system is recommended for low-slope roofs. Table 10.25 lists maximum spans for low-slope roof decks. Live loads can be determined from Table 10.24, and minimum fastener requirements are given in Table 10.26.

Rated Sheathing is equally effective under built-up roofing, asphalt or glass-fiber shingles, tile roofing; or wood shingles or shakes. Roof trusses spaced 24 in c to c are widely recognized as the most economical construction for residential roofs, particularly when $\frac{3}{8}$ - or $\frac{7}{16}$ -in, 24/0 sheathing with panel clips is used. However, use of fewer supports with thicker panels, for example, $\frac{23}{32}$ - or $\frac{3}{4}$ -in, 48/24 panels over framing 48 in c to c, is also cost-effective for long-span flat or pitched roofs. Live loads are given in Table 10.24. Nailing provisions are given in Table 10.26.

TABLE 10.23 Maximum Shear, lb/ft, for APA Panel Shear Walls for Wind or Seismic Loading^a
 (For framing of Douglas fir, larch, or southern pine)^b



Panel grade	Minimum nominal panel thickness, in	Minimum nail penetration in framing, in	Panels applied directly to framing				Panels applied over 1/2-in or 5/8-in gypsum sheathing					
			Nail size (common or galvanized box)	Nail spacing at panel edges, in			Nail size (common or galvanized box)	Nail spacing at panel edges, in				
				6	4	3		2 ^c	6	4	3	2 ^c
	5/16	1 1/4	6d	200	300	390	510	8d	200	300	390	510
APA Structural I grades	3/8	1 1/2	8d	230 ^d	366 ^d	460 ^d	610 ^d	10d ^e	280	430	550	730
	7/16			255 ^d	395 ^d	505 ^d	670 ^d					
	15/32			280	430	550	730					
	15/32	1 5/8	10d ^e	340	510	665	870					

Panel grade	Minimum nominal panel thickness, in	Minimum nail penetration in framing, in	Panels applied directly to framing				Panels applied over 1/2-in or 5/8-in gypsum sheathing					
			Nail size (common or galvanized box)	Nail spacing at panel edges, in				Nail size (common or galvanized box)	Nail spacing at panel edges, in			
				6	4	3	2 ^c		6	4	3	2 ^c
	5/16 or 1/4 ^f	1 1/4	6d	180	270	350	450	8d	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
APA Rated Sheathing; APA Rated Siding and other APA grades ^g except species Group 5	3/8	1 1/2	8d	220 ^d	320 ^d	410 ^d	530 ^d	10d ^e	260	380	490	640
	7/16			240 ^d	350 ^d	450 ^d	585 ^d					
	15/32			260	380	490	640					
	15/32	1 5/8	10d ^e	310	460	600	770					
19/32	340			510	665	870						
APA Rating Siding and other APA grades ^g except species Group 5			Nail size (galvanized casing)					Nail size (galvanized casing)				
	5/16 ^f	1 1/4	6d	140	210	275	360	8d	140	210	275	360
	3/8	1 1/2	8d	160	240	310	410	10d	160	240	310	410

^aAll panel edges should be backed with framing, which should have a nominal width of 2 in or more. The plywood may be installed horizontally or vertically. Space nails 6 in c to c along intermediate framing members for 3/8-in and 7/16-in panels attached to studs that are spaced 24 in c to c. For other conditions and panel thicknesses, space nails 12 in c to c on intermediate supports.

^bFor framing of other species, determine the species group of the lumber from the AF&PA National Design Specification for Wood Construction. Then, proceed as follows: For common or galvanized box nails, find the shear value from the above table for the nail size for Structural I panels, regardless of the actual grade. For galvanized casing nails, use the shear value given in the table. Next, multiply this value by 0.82 for Lumber Group III or 0.65 for Lumber Group IV.

^cFraming at adjoining edge panels should have a nominal width of 3 in or more and nails should be staggered.

^dIf studs are spaced at most 16 in c to c or if panels are applied with the long dimension across the studs, shears may be increased to the values shown for 15/32-in-thick sheathing with the same nail spacing.

^eFraming at adjoining panels should have a nominal width of 3 in or more and nails with a penetration into the framing exceeding 1 5/8-in and 3-in spacing c to c should be staggered.

^fFor 3/8-in-thick siding or APA Rated Siding when used as exterior siding applied directly to the framing, stud spacing of 16 in c to c is recommended.

^gValues apply to all-veneer plywood APA Rated Siding panels only. For APA Rated Siding on framing spaced 16 in c to c, the plywood may be 11/32 in thick, 3/8 in thick, or thicker. Thickness at the point of nailing on panel edges governs shear values.

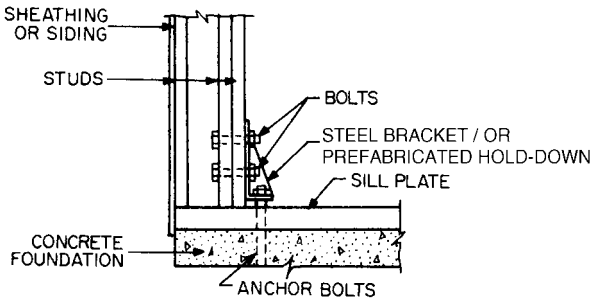


FIGURE 10.10 Foundation anchor for a wood shear wall.

TABLE 10.24 Maximum Uniform Roof Live Loads, psf,^a for APA Rated Sheathing^b and APA Rated Sturd-I-Floor
(Long dimension perpendicular to supports)^c

(a) APA Rated Sheathing ^b												
Panel span rating	Minimum panel thickness, in	Maximum span, in		Spacing of supports, in, c to c								
		With edge support ^d	Without edge support	12	16	20	24	32	40	48	60	
12/0	5/16	12	12	30								
16/0	5/16	16	16	70	30							
20/0	5/16	20	20	120	50	30						
24/0	3/8	24	20 ^e	190	100	60	30					
24/16	7/16	24	24	190	100	65	40					
³² / ₁₆	15/32, 1/2	32	28	325	180	120	70	30				
40/20	19/32, 5/8	40	32		305	205	130	60	30			
48/24	23/32, 3/4	48	36			280	175	95	45	35		

(b) APA Rated Sturd-I-Floor												
Stud spacing, in	Minimum panel thickness, in	Maximum span, in		Spacing of supports, in, c to c								
		With edge support	Without edge support	12	16	20	24	32	40	48	60	
16 oc	19/32, 5/8	24	24	185	100	65	40					
20 oc	19/32, 5/8	32	32	270	150	100	60	30				
24 oc	23/32, 3/4	40	36		240	160	100	50	30	25		
32 oc	7/8	48	40			295	185	100	60	40		
48 oc	13/32, 1 1/8	60	48				290	160	100	65	40	

^aMaximum loads include an assumed 10 psf for dead load.

^bIncludes APA Rated Sheathing Ceiling Deck.

^cApplies to panels 24 in or more wide.

^dEdge support is provided by such means as tongue-and-groove edges, panel edge clips (generally one midway between each support but two equally spaced between supports that are 48 in c to c), or lumber or other blocking.

^eMaximum span is 24 in for 15/32-in and 1/2-in panels.

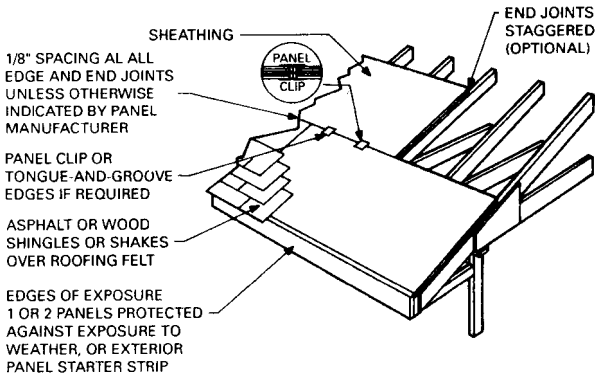


FIGURE 10.11 Structural panels installed as roof sheathing.

TABLE 10.25 Maximum Spans for APA Panel Roof Decks for Low-Slope Roofs*
(Panels set with long dimension perpendicular to supports and continuous over two or more spans)

Grade	Minimum nominal panel thickness, in	Minimum span rating	Maximum span, in	Number of panel clips per span [†]
APA Rated Sheathing	$1\frac{5}{32}$, $\frac{1}{2}$	32/16	24	1
	$1\frac{9}{32}$, $\frac{5}{8}$	40/20	32	1
	$2\frac{2}{32}$, $\frac{3}{4}$	48/24	48	2

* Built-up, single-ply, or modified bitumen roofing systems may be used for low-slope roofs. For guaranteed or warranted roofs, check with the membrane manufacturer for requirements for an acceptable deck.

[†] Edge support may also be provided by tongue-and-groove edges or solid blocking.

When support spacing exceeds the maximum length of an unsupported edge, as given in Table 10.24, provide adequate block, tongue-and-groove edges, or other edge support such as panel clips. Some types of panel clips, in addition to edge support, automatically assure recommended panel spacing. When required, use one panel clip per span of less than 28 in and two for 48-in or longer spans.

10.12.12 Preframed Roof Panels

Spans of 8 to 12 ft are usually the most practical with preframed panel construction which are typically used in large low slope roof applications, although spans up to 30 ft are not uncommon. Unsanded 4×8 -ft panels with stiffeners preframed at 16 or 24 in c to c are common. The long dimension of panels typically runs parallel to supports. Stiffeners and roof purlins provide support for all panel edges. Minimum nailing requirements for preframed panels are the same as for roof sheathing.

For preframed panels 8×8 ft or larger, the long panel dimension may run either parallel or perpendicular to stiffeners spaced 16 or 24 in c to c. Placing the long dimension across supports may require edge support such as panel clips or cleats between stiffeners at midspan in accordance with Table 10.24.

TABLE 10.26 Minimum Fastening for APA Panel Roof Sheathing (increased nail schedules may be required in high wind areas)

Panel thickness, [†] in	Nailing*		
	Size	Spacing, in	
		Panel edge	Intermediate
$\frac{5}{16}$ to 1	8d	6	12 $\frac{1}{2}$
1 $\frac{1}{8}$	8d or 10d	6	12 $\frac{1}{2}$

*In general, use common smooth or deformed-shank nails for panels up to 1 in thick. For 1 $\frac{1}{8}$ -in-thick panels, use 8d ring- or screw-shank or 10d common smooth-shank nails. Other approved fasteners, however, may be used.

[†]For stapling asphalt shingles to panels $\frac{5}{16}$ in or more thick, use staples with a $\frac{15}{16}$ -in minimum crown width and a 1-in-long leg. Space the staples in accordance with the recommendations of the shingle manufacturer.

$\frac{1}{2}$ For spans of 48 in or more, space the nails 6 in c to c at all supports.

Deflection limits are 1/180 of the span for total load; 1/240 for live load only. Nailing requirements for preframed panels are the same as for roof sheathing. See “APA Design/Construction Guide—Residential and Commercial,” APA—The Engineered Wood Association, for recommended maximum roof loads.

10.12.13 Panel Diaphragms

With only slight design modifications, any panel roof-deck system described in Arts. 10.12.11 and 10.12.12 will also function as an engineered roof or floor diaphragm to resist wind and seismic loading.

The ability of a diaphragm to function effectively as a deep beam, transferring lateral loads to shear walls, is related to the quality of the connections. Nailing is critical, since shears are transmitted through these fasteners. Common nails provide required strength. Other nail types may be used when their lateral bearing values are considered in the design. Load-carrying capacity is highest when the diaphragm is blocked.

Where 1 $\frac{1}{8}$ -in roof panels are desired, such as for heavy timber construction, shear values for $\frac{19}{32}$ -in panels are used. Blocked shear values for 1 $\frac{1}{8}$ -in panels may be obtained by specifying stapled tongue-and-groove (T&G) edges. Staples should be 16 ga, 1 in long, with a $\frac{3}{8}$ -in crown. They should be driven through the T&G edges $\frac{3}{8}$ -in from the joint so as to penetrate the tongue. Staples should be spaced at one-half of the boundary nail spacing for Cases 1 and 2, and at one-third the boundary nail spacing for Case 3 through 6, as illustrated in Table 10.27, which gives panel and fastening recommendations for roof diaphragms. Panels and framing are assumed already designed for perpendicular loads. To design a diaphragm, follow these steps:

1. Determine lateral loads and resulting shears.
2. Determine the nailing schedule (Table 10.27). Consider the load direction with respect to joints.

3. Compute the chord stress due to bending moment. Provide adequate splices. Check deflections. Check the anchorage of boundary framing, for example, to walls.

For situations where greater diaphragm capacities are necessary and framing with a nominal thickness of 3 or 4 in is available, diaphragms may be constructed using heavier nailing schedules, such as that given in the Uniform Building Code. (“Diaphragms,” APA—The Engineered Wood Association, Tacoma, Wash.)

10.13 DESIGN VALUES FOR MECHANICAL CONNECTIONS

Nails, staples, spikes, wood screws, bolts, and timber connectors, such as shear plates and split rings, are used for connections in wood construction. Because determination of stress distribution in connections made with wood and metal is complicated, information for design of joints has been developed from tests and experience. The data indicate that design values and methods of design for mechanical connections are applicable to both solid sawn lumber and laminated members. The “National Design Specification for Wood Construction,” (NDS) American Forest & Paper Association, provides design equations and tabulates design values for connections made with various types of fasteners. Design values for connections made with more than one type of fastener, however, should be based on tests or special analysis.

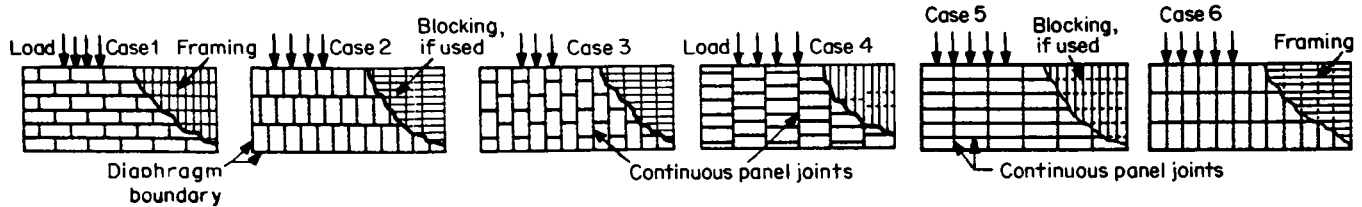
10.14 ADJUSTMENT OF DESIGN VALUES FOR CONNECTIONS

Nominal design values for laterally loaded fasteners Z , withdrawal of fasteners W , load parallel to grain P , and load perpendicular to grain Q should be multiplied by applicable adjustment factors to determine adjusted design values Z' , W' , P' , and Q' , respectively. Table 10.28 summarizes the adjustment factors that should be applied to the design values Z , W , P , and Q for connections made with commonly used types of fasteners. The load applied to a connection should not exceed the adjusted design value.

10.14.1 Load Duration Factor

Nominal design values should be multiplied by the load duration factor C_D specified in Art. 10.5.1, except that C_D may not exceed 1.6 or when the capacity of the connection is controlled by the strength of metal. The impact load duration factor shall not apply to connections.

TABLE 10.27 Maximum Shear, lb/ft, for APA Panel Diaphragms for Wind or Seismic Loading
(For Framing of Douglas fir, larch, or southern pine)*



Panel grade	Common nail size	Minimum nail penetration in framing, in	Minimum nominal panel thickness, in	Minimum nominal width of framing member, in	Blocked diaphragms				Unblocked diaphragms	
					Nail spacing, in, at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3, 4), and at all panel edges (Cases 5, 6)†				Nails spaced 6 in max at supported edges†	
					6	4	2½‡	2‡	Case 1 (no unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5, 6)
					Nail spacing, in, at other panel edges (Cases 1, 2, 3, 4)					
6	6	4	3	6	6	4	3	6	6	
APA Structural I grades	6d	1¼	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1½	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d§	1⅝	15/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	250

Panel grade	Common nail size	Minimum nail penetration in framing, in	Minimum nominal panel thickness, in	Minimum nominal width of framing member, in	Nail spacing, in, at other panel edges (Cases 1, 2, 3, 4)				Case 1 (No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5, 6)	
					6	6	4	3			
APA Rated Sheathing, APA Rated Sturd-I-Floor and other APA grades except Species Group 5	6d	1¼	5/16	2	170	225	335	380	150	110	
				3	190	250	380	430	170	125	
				3/8	2	185	250	375	420	165	125
					3	210	280	420	475	185	140
	8d	1½	3/8	2	240	320	480	545	215	160	
				3	270	360	540	610	240	180	
				7/16	2	255	340	505	575	230	170
	3	285	380		570	645	255	190			
10d§	1⅝	15/32	2		360	530	600	240	180		
			3		400	600	675	265	200		
			19/32	2	320	425	640	730	285	215	
				3	360	480	720	820	320	240	

*For framing of other species, determine the species group of the lumber from the AF&PA National Design Specification for Wood Construction. Then, proceed as follows: Find the shear value from the above table for the nail size for Structural I panels, regardless of the actual grade. Multiply this value by 0.82 for Lumber Group III or 0.65 for Lumber Group IV.

† In general, space nails 12 in c to c along intermediate framing members, but use 6 in when supports are 48 in c to c. Local building codes may require 10-in nail spacing at intermediate supports for floors.

‡ Framing at adjoining panel edges should have a nominal width of 3 in or more and nails should be staggered.

§ Framing at adjoining panel edges should have a nominal width of 3 in or more and nails with a penetration into the framing exceeding 1⅝ in and 3-in spacing c to c should be staggered.

Note: Designs based on diaphragm stresses depend on the direction of continuous panel joints with respect to the loads, rather than on the directions of the long dimension of the panels. For blocked diaphragms, continuous framing may be in either direction.

TABLE 10.28 Adjusted Design Values for Connections*

Bolts:

$$Z' = ZC_D C_M C_t C_g C_\Delta$$

Split-ring and shear-plate connectors:

$$P' = PC_D C_M C_t C_g C_\Delta C_d C_{st}$$

$$Q' = QC_D C_M C_t C_g C_\Delta C_d$$

Lag screws:

$$W' = WC_D C_M C_t C_{eg}$$

$$Z' = ZC_D C_M C_t C_g C_\Delta C_d C_{eg}$$

Wood screws:

$$W' = WC_D C_M C_t$$

$$Z' = ZC_D C_M C_t C_d C_{eg}$$

Nails and spikes:

$$W' = WC_D C_M C_t C_m$$

$$Z' = ZC_D C_M C_t C_d C_{eg} C_{di} C_m$$

Metal plate connectors:

$$Z' = ZC_D C_M C_t$$

Drift bolts and drift pins:

$$W' = WC_D C_M C_t C_{eg}$$

$$Z' = ZC_D C_M C_t C_g C_\Delta C_d C_{eg}$$

Spike grids:

$$Z' = ZC_D C_M C_t C_\Delta$$

Timber rivets:

$$P' = PC_D C_M C_t C_{st}$$

$$Q' = QC_D C_M C_t C_\Delta C_{st}$$

*The adjustment factors are as follows:

C_D = load duration factor, not to exceed 1.6 for connections (Art. 10.14.1)

C_M = wet-service factor, not applicable to toe-nails loaded in withdrawal (Art. 10.14.2)

C_t = temperature factor (Art. 10.14.3)

C_g = group-action factor (Art. 10.14.4)

C_Δ = geometry factor (Art. 10.14.5)

C_d = penetration-depth factor (Art. 10.14.6)

C_{eg} = end-grain factor (Art. 10.14.7)

C_{st} = metal-side-plate factor (Art. 10.14.8)

C_{di} = diaphragm factor (Art. 10.14.9)

C_m = toenail factor (Art. 10.14.10)

10.14.2 Wet-Service Factor

Nominal design values apply to wood that will be used under dry-service conditions; that is, where moisture content of the wood will be a maximum of 19% of the oven-dry weight, as would be the case in most covered structures. For connections in wood that is unseasoned or partly seasoned, or when connections will be exposed to wet-service conditions in use, nominal design values should be multiplied by the appropriate wet-service factor C_M in Table 10.29.

TABLE 10.29 Wet Service Factors, C_M , for Connections

Fastener type	Moisture content			
	At time of fabrication	In-service	Load	
			Lateral	Withdrawal
Shear Plates Split Rings ¹	$\leq 19\%$	$\leq 19\%$	1.0	—
	$> 19\%$	$\leq 19\%$	0.8	—
	any	$> 19\%$	0.7	—
Metal Connector Plates	$\leq 19\%$	$\leq 19\%$	1.0	—
	$> 19\%$	$\leq 19\%$	0.8	—
	any	$> 19\%$	0.8	—
Bolts & Drift Pins & Drift Bolts	any	$\leq 19\%$	1.0 ²	—
	any	$> 19\%$	0.7	—
Lag Screws & Wood Screws	any	$\leq 19\%$	1.0 ³	1.0
	any	$> 19\%$	0.7	0.7
Nails & Spikes	$\leq 19\%$	$\leq 19\%$	1.0	1.0
	$> 19\%$	$\leq 19\%$	0.7	0.25
	$\leq 19\%$	$> 19\%$	0.7	0.25
	$> 19\%$	$> 19\%$	0.7	1.0
Threaded Hardened Nails	any	any	1.0	1.0
Timber Rivets	$\leq 19\%$	$\leq 19\%$	1.0	—
	$> 19\%$	$\leq 19\%$	0.9	—
	$\leq 19\%$	$> 19\%$	0.8	—
	$> 19\%$	$> 19\%$	0.8	—

1. For split ring or shear plate connectors, moisture content limitations apply to a depth of 3/4" below the surface of the wood.

2. Wood to wood connections involving metal connector plates for load transfers shall be designed in accordance with ANSI /TPI 1-1995.

3. For two or more rows of bolts or lag screws with single steel side plate(s) installed in wood with moisture content $> 19\%$ at time of fabric service, $C_M = 0.4$.

10.14.3 Temperature Factor

Nominal design values should be multiplied by the appropriate temperature factor C_t listed in Table 10.30 for connections that will experience sustained exposure to elevated temperatures up to 150°F.

10.14.4 Group-Action Factor

Nominal design values for split-ring connectors, shear-plate connectors, bolts with diameter D up to 1 in, and lag screws in a row should be multiplied by the group-action factor C_g given in Table 10.31. The NDS contains design criteria for determination of C_g for additional configurations.

For determination of C_g , a row of fasteners is defined as any of the following:

TABLE 10.30 Temperature Factor C_t for Connections

In-service moisture conditions*	$T \leq 100^\circ\text{F}$	$100^\circ\text{F} < T \leq 125^\circ\text{F}$	$125^\circ\text{F} < T \leq 150^\circ\text{F}$
Dry	1.0	0.8	0.7
Wet	1.0	0.7	0.5

* Wet and dry service conditions are defined in a Table 10.29 footnote.

1. Two or more split-ring or shear-plate connectors aligned with the direction of the load.
2. Two or more bolts with the same diameter, loaded in shear, and aligned with the direction of the load.
3. Two or more lag screws of the same type and size loaded in single shear and aligned with the direction of the load.

When fasteners in adjacent rows are staggered but close together, they may have to be treated as a single row in determination of C_g . This occurs when the distance between adjacent rows is less than one-fourth of the spacing between the closest fasteners in adjacent rows.

The group-action factor is necessary because of the following characteristics of a joint with more than two fasteners in a row: The two end fasteners carry a larger load than the interior fasteners. With six or more fasteners in a row, the two end fasteners may carry more than 50% of the load. With bolts, a small redistribution of load from the end bolts to the interior bolts occurs due to crushing of the wood at the end bolts. If failure is in shear, a partial failure occurs before substantial redistribution of load takes place.

10.14.5 Geometry Factor

The NDS specifies minimum edge distance, end distance, and spacing required for full design value for bolts, lag screws, and split-ring and shear-plate connectors. The NDS also tabulates nominal design values for these fasteners based on the minimum distances. When the end distance or the spacing is less than the minimum required for full design value but larger than the minimum required for reduced design value, nominal design values should be multiplied by the smallest applicable geometry factor C_Δ determined from the end distance and spacing requirements for the type of connector specified. The smallest geometry factor for any connector in a group should be applied to all in the group. For multiple shear connections or for asymmetric three-member connections, the smallest geometry factor for any shear plane should be applied to all fasteners in the connection. The NDS also outlines geometry factors for timber rivet connections loaded perpendicular to grain.

10.14.6 Penetration-Depth Factor

Nominal lateral design values for lag screws, wood screws, nails, and spikes are based on a specific penetration into the main member, as established by the shank

TABLE 10.31 Group-Action Factors

(a) For bolt or lag-screw connections with wood side members*												
A_s/A_m^\dagger	A_s, \ddagger in ²	Number of fasteners in a row										
		2	3	4	5	6	7	8	9	10	11	12
0.5	5	0.98	0.92	0.84	0.75	0.68	0.61	0.55	0.50	0.45	0.41	0.38
	12	0.99	0.96	0.92	0.87	0.81	0.76	0.70	0.65	0.61	0.57	0.53
	20	0.99	0.98	0.95	0.91	0.87	0.83	0.78	0.74	0.70	0.66	0.62
	28	1.00	0.98	0.96	0.93	0.90	0.87	0.83	0.79	0.76	0.72	0.69
	40	1.00	0.99	0.97	0.95	0.93	0.90	0.87	0.84	0.81	0.78	0.75
	64	1.00	0.99	0.98	0.97	0.95	0.93	0.91	0.89	0.87	0.84	0.82
1	5	1.00	0.97	0.91	0.85	0.78	0.71	0.64	0.59	0.54	0.49	0.45
	12	1.00	0.99	0.96	0.93	0.88	0.84	0.79	0.74	0.70	0.65	0.61
	20	1.00	0.99	0.98	0.95	0.92	0.89	0.86	0.82	0.78	0.75	0.71
	28	1.00	0.99	0.98	0.97	0.94	0.92	0.89	0.86	0.83	0.80	0.77
	40	1.00	1.00	0.99	0.98	0.96	0.94	0.92	0.90	0.87	0.85	0.82
	64	1.00	1.00	0.99	0.98	0.97	0.96	0.95	0.93	0.91	0.90	0.88
(b) For 4-in split-ring or shear-plate connectors with wood side members§												
A_s/A_m^\dagger	A_s, \ddagger in ²	Number of fasteners in a row										
		2	3	4	5	6	7	8	9	10	11	12
0.5	5	0.90	0.73	0.59	0.48	0.41	0.35	0.31	0.27	0.25	0.22	0.20
	12	0.95	0.83	0.71	0.60	0.52	0.45	0.40	0.36	0.32	0.29	0.27
	20	0.97	0.88	0.78	0.69	0.60	0.53	0.47	0.43	0.39	0.35	0.32
	28	0.97	0.91	0.82	0.74	0.66	0.59	0.53	0.48	0.44	0.40	0.37
	40	0.98	0.93	0.86	0.79	0.72	0.65	0.59	0.54	0.49	0.45	0.42
	64	0.99	0.95	0.91	0.85	0.79	0.73	0.67	0.62	0.58	0.54	0.50
1	5	1.00	0.87	0.72	0.59	0.50	0.43	0.38	0.34	0.30	0.28	0.25
	12	1.00	0.93	0.83	0.72	0.63	0.55	0.48	0.43	0.39	0.36	0.33
	20	1.00	0.95	0.88	0.79	0.71	0.63	0.57	0.51	0.46	0.42	0.39
	28	1.00	0.97	0.91	0.83	0.76	0.69	0.62	0.57	0.52	0.47	0.44
	40	1.00	0.98	0.93	0.87	0.81	0.75	0.69	0.63	0.58	0.54	0.50
	64	1.00	0.98	0.95	0.91	0.87	0.82	0.77	0.72	0.67	0.62	0.58

*For fastener diameter $D = 1$ in and fastener spacing $s = 4$ in in bolt or lag-screw connections with modulus of elasticity for wood $E = 1,400,000$ psi. Tabulated values of C_g are conservative for $D < 1$ in, $s < 4$ in, or $E > 1,400,000$ psi.

$\dagger A_s$ = cross sectional area of the main members before boring or grooving and A_m = sum of the cross-sectional areas of the side members before boring or grooving. When $A_s/A_m > 1$, uses A_m/A_s .

$\ddagger A_s/S_m > 1$, use A_m instead of A_s .

§For spacing $s = 9$ in in connections made with 4-in split rings or shear plates with modulus of elasticity for wood $E = 1,400,000$ psi. Tabulated values of C_g are conservative for 2½-in split-ring connectors, 2½-in shear-plate connectors, $s < 9$ in, or $E > 1,400,000$ psi.

diameter. The NDS also set a minimum penetration into the main member for a reduced design value for each fastener. When the penetration is larger than the minimum but less than that assumed in establishment of the full lateral design value, linear interpolation should be used in determination of the penetration-depth factor C_d . In no case should C_d exceed unity. Table 10.32 lists minimum penetration and assumed penetration for full design value as well as C_d for each type of fastener.

TABLE 10.32 Penetration-Depth Factor*

Penetration, p	Lag screws	Wood screws	Nails or spikes
For full design value	$8D$	$7D$	$12D$
Minimum p	$4D$	$4D$	$6D$
C_d	$p/8D$	$p/7D$	$p/12D$

* D = fastener diameter.

10.14.7 End-Grain Factor

Woods screws, lag screws, nails, and spikes are used in two types of connections, withdrawal and lateral load. In withdrawal connections, the load is applied parallel to the length of the fastener. In laterally loaded connections, the load is applied perpendicular to the length of the fastener. Either type of connection is weaker when fasteners are inserted in the ends of a member, parallel to the grain, rather than in the side grain.

Withdrawal Design Value. Wood screws, nails, and spikes should not be loaded in withdrawal from end grain. Tests show that splitting of the wood member causes erratic results relative to those for withdrawal from side grain.

When lag screws are loaded in withdrawal from end grain, the nominal withdrawal design value should be multiplied by the end-grain factor $C_{cg} = 0.75$.

Lateral Design Value. When lag screws, wood screws, nails, or spikes are inserted, with the axis in the direction of the wood grain, into the end grain of a main member, the nominal design value for lateral loading should be multiplied by the end-grain factor $C_{cg} = 0.67$.

10.14.8 Metal-Side-Plate Factor

For timber rivets and 4" shear plates, larger design values are permitted when metal side plates are used in lieu of wood plates. For 4-in shear-plate connectors, the nominal design value for load parallel to grain P should be multiplied by the appropriate C_{st} given in Table 10.33(a). The values depend on the species of wood used in the connection, such as group A, B, C, or D listed in the NDS. Table 10.33(b) outlines metal side plate factors for timber rivet connections.

TABLE 10.33(a) Metal-Side-Plate Factors for Shear-Plate Connectors*

Species group†	C_{st}
A	1.18
B	1.11
C	1.05
D	1.00

* For 4-in shear plates loaded parallel to grain.

† For components of each species group, see the groupings in the NDS.

10.14.9 Diaphragm Factor

A diaphragm is a large, thin structural element that is loaded in its plane. When nails or spikes are used in a diaphragm connection, the nominal lateral design value should be multiplied by the diaphragm factor $C_{di} = 1.1$.

10.14.10 Toenail Factor

For such connections as stud to plate, beam to plate, and blocking to plate, toenailing is generally used. The NDS recommends that toenails be driven at an angle of about 30° with the face of the stud, beam, or blocking and started about one-third the length of the nail from the end of the member. For toenailed connections, the nominal lateral design values for connections with nails driven into side grain should be multiplied by the toenail factor $C_{tn} = 0.83$.

10.14.11 Adjustments for Fire-Retardant Treatment

For connections made with lumber or other wood products pressure-treated with fire-retardant chemicals, design values should be obtained from the company providing the treatment and redrying service. The load-duration factor for impact does not apply to such connections.

10.15 BOLTS

The “National Design Specification for Wood Construction,” American Forest & Paper Association, contains design provisions and design values for bolts with diameters up to 1 in conforming to the ANSI/ASME Standard B18.2.1. Bolt design values in the NDS apply to connections that have been snugly tightened and to connections that have loosened due to shrinkage of the wood components.

Following are some important NDS requirements for bolts: Bolt holes should have a diameter from $\frac{1}{32}$ to $\frac{1}{16}$ in larger than the bolt diameter. In establishment of design values, careful centering of holes in main members is assumed. Tight fit of bolts in the holes, requiring forced insertion, is not recommended. A metal plate, strap, or washer (not smaller than a standard cut washer) should be placed between the wood and bolt head and between the wood and the nut. The length of bolt threads subject to bearing on the wood should be kept to a practical minimum.

Two or more bolts placed in a line parallel to the direction of the load constitute a row. End distance is the minimum distance from the end of a member to the center of the bolt hole that is nearest to the end. Edge distance is the minimum distance from the edge of a member to the center of the nearest bolt hole. Figure 10.12 illustrates these distances, the spacing between rows, and the spacing of bolts in a row. NDS requirements are listed for minimum end distance in Table 10.34, for minimum edge distance in Table 10.35, and for minimum spacing between rows and between bolts in a row in Table 10.36. The geometry factor C_{Δ} discussed in Art. 10.14.5 is applied to the design value for a bolted connection when the end distance or spacing between bolts is less than that given in these tables for full design value.

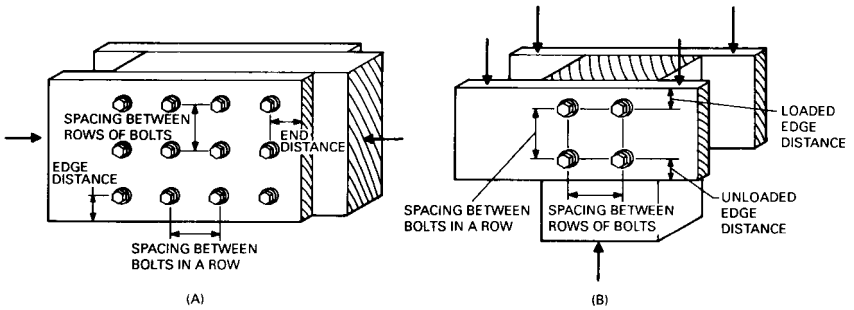


FIGURE 10.12 Bolt spacing and edge and end distances are defined with respect to load direction: (a) load parallel to the grain; (b) perpendicular to the grain. Reprinted with permission from the American Forest and Paper Association.

TABLE 10.34 Minimum End Distances for Bolts*

Direction of loading	For reduced design value	For full design value
Perpendicular to grain	2D	4D
Compression parallel to grain (bolt bearing away from member end)	2D	4D
Tension parallel to grain (bolt bearing toward member end):		
For softwoods	3.5D	7D
For hardwoods	2.5D	5D

*D = bolt diameter

TABLE 10.35 Minimum Edge Distance for Bolts

Direction of loading*	Minimum edge distance
Parallel to grain:	
When $L/D \leq 6$	1.5D
When $L/D > 6$	1.5D or half the spacing between rows, whichever is greater
Perpendicular to grain:	
To loaded edge	4D
To unloaded edge	1.5D

*L = length of bolt in main member and D = bolt diameter.

10.16 LAG SCREWS

Also known as lag bolts, lag screws are large screws with a square or hexagonal bolt head. As is the case with bolts and timber connectors, lag screws are used where relatively heavy loads have to be transmitted in a connection. They are used in lieu of bolts where the components of a joint are so thick that an excessively

TABLE 10.36 Minimum Spacing for Bolts*

(a) For bolts in a row		
Direction of loading	For reduced design value	For full design value
Parallel to grain	$3D$	$4D$
Perpendicular to grain	$3D$	Required spacing for attached member(s)
(b) Between bolts in a row		
Direction of loading	Minimum spacing	
Parallel to grain	$1.5D$	
Perpendicular to grain:		
When $L/D \leq 2$	$2.5D$	
When $2 < L/D < 6$	$(5L + 10D)/8$	
When $L/D \geq 6$	$5D$	

* L = length of bolt in main member and D = bolt diameter.

long bolt would be needed, where one side of a connection is not accessible, or where heavy withdrawal loads have to be resisted. If desired, lag screws can be used in conjunction with split rings and shear plates.

Lag screws are turned with a wrench into prebored holes with total length equal to the nominal screw length. Soap or other lubricant may be used to facilitate insertion and prevent damage to screws. Two holes are drilled for each lag screw. The first and deepest hole has a diameter, as specified in the NDS for various species, depending on the wood density, ranging from 40 to 85% of the shank diameter. The second hole should have the same diameter as the shank, or unthreaded portion of the lag screw, and the same depth as the unthreaded portion.

The NDS contains design values and design provisions for lag screws that conform to ANSI/ASME Standard B18.2.1. Lag screws loaded in withdrawal should be designed for allowable tensile strength in the net (root-of-thread) section as well as for resistance to withdrawal. For single-shear wood-to-wood connections, the lag screw should be inserted in the side grain of the main member with the screw axis perpendicular to the wood fibers. Minimum edge and end distances and spacing for a lag screw are the same as for a bolt with diameter equal to the shank diameter of the screw (Tables 10.34 to 10.36).

10.17 SPLIT-RING AND SHEAR-PLATE CONNECTORS

These are metal devices used with bolts or lag screws for producing joints with fewer fasteners without reduction in strength. Several types of connectors are available. Usually, they are either steel rings, called split rings, that are placed in grooves in adjoining members to prevent relative movement or metal plates, called shear

plates, embedded in the faces of adjoining timbers. The bolts or lag screws are used with these connectors to prevent the timbers from separating. The load is transmitted across the joint through the connectors.

Split rings are used for joining wood to wood. They are placed in circular grooves cut by a hand tool in the contact surfaces. About half the depth of each ring is in each of the two members in contact. A bolt hole is drilled through the center of the core encircled by the groove. For economic reasons, split rings are seldom used now because of the accuracy required for properly fabricating the wood members and the relative difficulty of installation.

A single shear plate is used for wood-to-steel connections (Fig. 10.13*a* and *b*). When used in pairs, split rings may be used for wood-to-wood connections (Fig. 10.13*c*). Set with one plate in each member at the contact surface, they enable the members to slide easily into position during fabrication of the joint, thus reducing the labor needed to make the connection. Shear plates are placed in precut daps and are completely embedded in the timber, flush with the surface. As with split rings, the role of the bolt or lag screw through each plate is to prevent the components of the joint from separating; loads are transmitted across the joint through the plates. They are manufactured in 2 $\frac{5}{8}$ - and 4-in diameter.

Shear plates are useful in demountable structures. They may be installed in the members immediately after fabrication and held in position by nails.

Toothed rings and spike grids sometimes are used for special applications; but shear plates and occasionally split rings are the prime connectors for joints in timber construction requiring transmission of very heavy loads.

The NDS contains tables that give design values for shear-plate connections. The tabulated values apply to seasoned timbers used where they will remain dry. See Table 10.29 for wet service factors.

Design values for split rings and shear plates for angles between 0° (parallel to grain) and 90° (perpendicular to grain) may be obtained from the Hankinson formula:

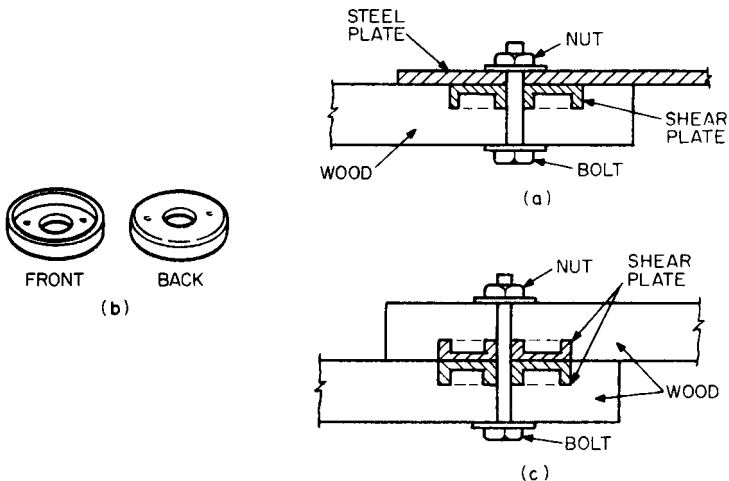


FIGURE 10.13 Shear-plate connections. (a) A steel plate connected to a wood member with a shear-plate connector. (b) Shear-plate connector. (c) Two wood members connected with a pair of shear plates and a bolt.

$$N' = \frac{P'Q'}{P' \sin^2 \theta + Q' \cos^2 \theta} \quad (10.32)$$

where N' , P' , and Q' are, respectively, the adjusted design value, lb, or stress, psi, at inclination θ with the direction of grain, parallel to grain, and perpendicular to grain.

Design values are based on the assumption that the wood at the joint is clear and relatively free from checks, shakes, and splits. If knots are present in the longitudinal projection of the net section within a distance from the critical section of half the diameter of the connector, the area of the knot should be subtracted from the area of the critical section. It is assumed that slope of the grain at the joint does not exceed 1 in 10.

The stress, whether tension or compression, in the net area, the area remaining at the critical section after subtracting the projected area of the connectors and the bolt from the full cross-sectional area of the member, should not exceed the design value of clear wood in compression parallel to the grain.

Tables in the NDS list the least thickness of member that should be used with the various sizes of connectors. The design values listed for the greatest thickness of member with each type and size of connector unit are the maximums to be used for all thicker material. Design values for members with thicknesses between those listed may be obtained by interpolation.

The NDS also lists minimum end and edge distances and spacing for timber connectors (Table 10.37). Edge distance is the distance from the edge of a member to the center of the connector closest to that edge and measured perpendicular to the edge. End distance is measured parallel to the grain from the center of the connector to the square-cut end of the member. If the end of the member is not cut normal to the longitudinal axis, the end distance, measured parallel to that axis from any point on the center half of the connector diameter that is perpendicular to the axis, should not be less than the minimum end distance required for a square-cut member. Spacing of connectors is measured between their centers along a line between centers.

10.18 WOOD SCREWS

The "National Design Specification for Wood Construction" contains design provisions and design values for wood screws that conform to ANSI/ASME Standard B18.6.1. When wood screws are loaded in withdrawal, the design value for tension in the screws should not be exceeded. For single-shear wood-to-wood construction, the screws should be inserted in the side grain of the main member with axis perpendicular to the wood fibers.

Edge and end distances and spacing of wood screws should be sufficient to prevent splitting of the wood. If building-code requirements for such distances are not available, Table 10.38 may be used to establish wood-screw patterns. Spacing, or pitch, between fasteners in a row is affected by species, moisture content, and grain orientation.

Screws may be inserted in lumber or timber that has a specific gravity less than 0.50, without preboring a hole for them. In denser wood, lead holes should be drilled and screws inserted by turning, not by hammering. Holes for wood screws loaded in withdrawal should have a diameter of 90% of the screw root diameter in

TABLE 10.37 Minimum Edge and End Distances, Spacing, and Geometry Factors C_{Δ} for Split-Ring and Shear-Plate Connectors

		2½-in Split-ring connectors 2⅝-in Shear-plate connectors				4-in Split-ring connectors 4-in Shear-plate connectors			
		Loads parallel to grain		Loads perpendicular to grain		Load parallel to grain		Loads perpendicular to grain	
		For reduced design value	For full design value	For reduced design value	For full design value	For reduced design value	For full design value	For reduced design value	For full design value
Edge distance	Unloaded edge, in C_{Δ}	1¾ 1.0	1¾ 1.0	1¾ 1.0	1¾ 1.0	2¾ 1.0	2¾ 1.0	2¾ 1.0	2¾ 1.0
	Loaded edge, in C_{Δ}	1¾ 1.0	1¾ 1.0	1¾ 0.83	2¾ 1.0	2¾ 1.0	2¾ 1.0	2¾ 0.83	3¾ 1.0
End distance	Tension member, in C_{Δ}	2¾ 0.625	5½ 1.0	2¾ 0.625	5½ 1.0	3½ 0.625	7 1.0	3½ 0.625	7 1.0
	Compression member, in C_{Δ}	2½ 0.626	4 1.0	2¾ 0.625	5½ 1.0	3¼ 0.625	5½ 1.0	3½ 0.625	7 1.0
Spacing	Spacing parallel to grain, in C_{Δ}	3½ 0.5	6¾ 1.0	3½ 1.0	3½ 1.0	5 0.5	9 1.0	5 1.0	5 1.0
	Spacing perpendicular to grain, in C_{Δ}	3½ 1.0	3½ 1.0	3½ 0.5	4¼ 1.0	5 1.0	5 1.0	5 0.5	6 1.0

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TABLE 10.38 Recommended Minimum Edge and End Distances and Spacing for Wood Screws, Nails, and Spikes

	Wood side members	
	Not prebored	Prebored
Edge distance	$2.5d$	$2.5d$
End distance:		
For tension load parallel to grain	$15d$	$10d$
For compression load parallel to grain	$10d$	$5d$
Spacing (pitch) between fasteners in a row:		
Parallel to grain	$15d$	$10d$
Perpendicular to grain	$10d$	$5d$
Spacing (gage) between rows of fasteners:		
In-line	$5d$	$3d$
Staggered	$2.5d$	$2.5d$
	Steel side members	
	Not prebored	Prebored
Edge distance	$2.5d$	$2.5d$
End distance:		
For tension load parallel to grain	$10d$	$5d$
For compression load parallel to grain	$5d$	$3d$
Spacing (pitch) between fasteners in a row:		
Parallel to grain	$10d$	$5d$
Perpendicular to grain	$5d$	$2.5d$
Spacing (gage) between rows of fasteners:		
In-line	$3d$	$2.5d$
Staggered	$2.5d$	$2.5d$

* d = shank diameter.

such wood as oak, with a specific gravity of 0.6 or more, and of 70% of the root diameter in such wood as Douglas Fir and Larch, with a specific gravity between 0.5 and 0.6. For wood screws subjected to lateral loads, holes receiving the shanks in wood with a specific gravity of 0.6 or more should have the same diameter as the shank. Holes for the threaded portion should have a diameter about equal to the root diameter of the screws. For screws in less dense wood, the part of the hole for the shank should be about seven-eighths the diameter of the shank. Holes for the threaded portion should have a diameter about seven-eighths the root diameter. Soap or other lubricant may be used to facilitate insertion and to prevent damage to the screw.

Screws are designated by gage (diameter) of shank and overall length. For design purposes, it is adequate to assume two-thirds of the screw length is threaded.

10.19 NAILS AND SPIKES

The "National Design Specification for Wood Construction" (NDS) provides design values and design provisions for nails, spikes, box nails, and threaded, hardened-

steel nails and spikes conforming to ASTM F1667. Sizes of nails are specified by the pennyweight, indicated by d. The pennyweight for different types of nails establishes lengths, shank diameter, and head size of the nail. Common wire nails and spikes are about the same, except that spikes have a larger diameter than nails with the same pennyweight designation. Box nails have a smaller diameter than common nails. For a specific pennyweight, the five types of nails have the same length.

Edge and end distances and spacing for nails and spikes should be sufficient to prevent splitting of the wood. If specific code requirements for these distances are not available, Table 10.38 may be used to establish nailing patterns. Spacing, or pitch, between fasteners in a row is affected by species, moisture content, and grain orientation.

When a prebored hole is to be used to prevent splitting of the wood, the hole diameter should not exceed 90% of the nail or spike diameter for wood with a specific gravity exceeding 0.6, and should not exceed 75% of the diameter for less dense wood.

10.20 STRUCTURAL FRAMING CONNECTIONS

Standard and special preengineered metal hangers are used extensively in timber construction. Stock hangers are available from a number of manufacturers, but most manufacturers also provide hangers of special design. Where appearance is of prime importance, concealed hangers are frequently selected.

Figures 10.14 to 10.16 show typical structural framing details such as beam hangers and connectors and column anchors.

(See "Heavy Timber Connection Details," WCD No. 5, American Forest and Paper Association, Washington, D.C., for additional information.)

10.21 GLUED FASTENINGS

Glued joints are generally made between two pieces of wood where the grain directions are parallel (as between the laminations of a beam or arch). Or such joints may be between solid-sawn or laminated timber and plywood, where the face grain of the plywood is either parallel or at right angles to the grain direction of the timber.

It is only in special cases that lumber may be glued with the grain direction of adjacent pieces at an angle. When the angle is large, dimensional changes caused by variations in wood moisture content set up large stresses in the glued joint. Consequently, the strength of the joint may be considerably reduced over a period of time. Exact data are not available, however, on the magnitude of this expected strength reduction.

In joints connected with plywood gusset plates, this shrinkage differential is minimized, because plywood swells and shrinks much less than does solid wood.

Glued joints can be made between end-grain surfaces. They are seldom strong enough, however, to meet the requirements of even ordinary service. Seldom is it possible to develop more than 25% of the tensile strength of the wood in such butt joints. It is for this reason that plane sloping scarfs of relatively flat slope or finger

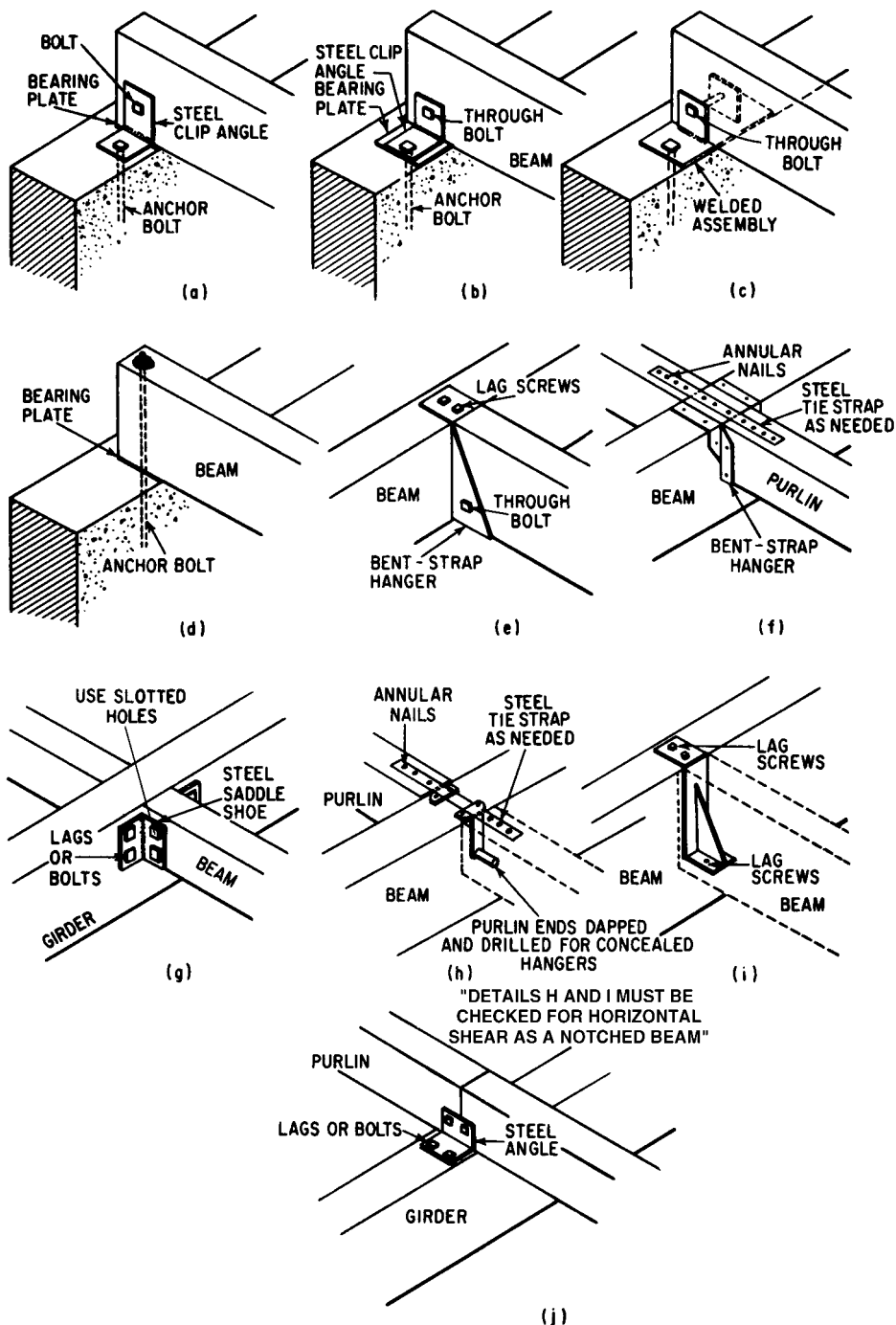


FIGURE 10.14 Beam connections: (a), (b) wood beam anchored on a wall with steel angles; (c) with welded assembly; (d) beam anchored directly with a bolt; (e) beam supported on a girder with bent-strap hanger; (f) similar support for purlins; (g) saddle connects beam to girder (suitable for one-sided connection); (h), (i) connections with concealed hangers; (j), connection with steel angles.

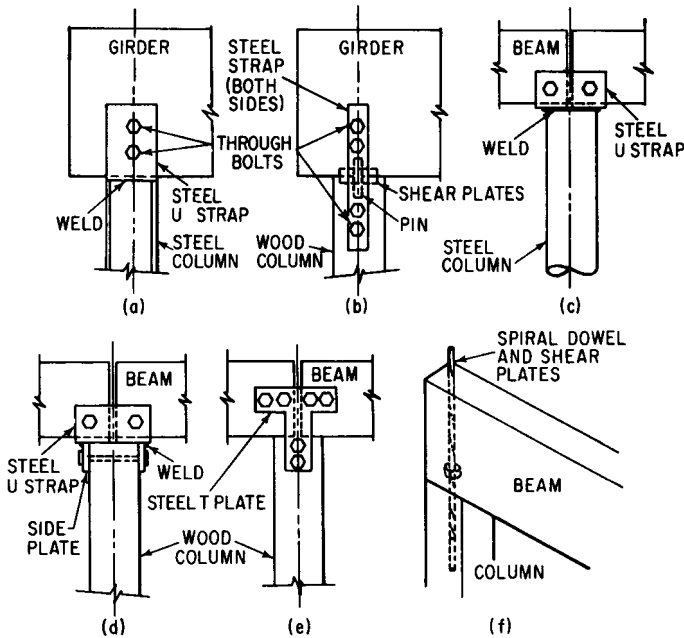


FIGURE 10.15 Typical wood beam and girder connections to columns: (a) wood girder to steel column; (b) girder to wood column; (c) beam to pipe column; (d) beam to wood column, with steel trap welded to steel side plates; (e) beam to wood column, with a T plate; (f) beam-column connection with spiral dowel and shear plates.

joints with thin tips and flat slope on the sides of the individual fingers are recommended to develop a high proportion of the strength of the wood.

Joints of end grain to side grain are also difficult to glue properly. When subjected to severe stresses as a result of unequal dimensional changes in the members due to changes in moisture content, joints suffer from severely reduced strength.

For the preceding reasons, joints between end-grain surfaces and between end-grain and side-grain surfaces should not be used if the joints are expected to carry load.

For joints made with wood of different species, the allowable shear stress for parallel-grain bonding is equal to the allowable shear stress parallel to the grain for the weaker species in the joint. This assumes uniform stress distribution in the joint. When grain direction is not parallel, the allowable shear stress on the glued area between the two pieces may be estimated from Eq. (10.32).

(See K. F. Faherty and T. G. Williamson, "Wood Engineering and Construction Handbook," McGraw-Hill Publishing Company, New York, for additional information.)

10.22 WOOD TRUSSES

Used in all forms of building construction for centuries, wood trusses have appeared in a wide variety of forms. They utilized many different wood and nonwood struc-

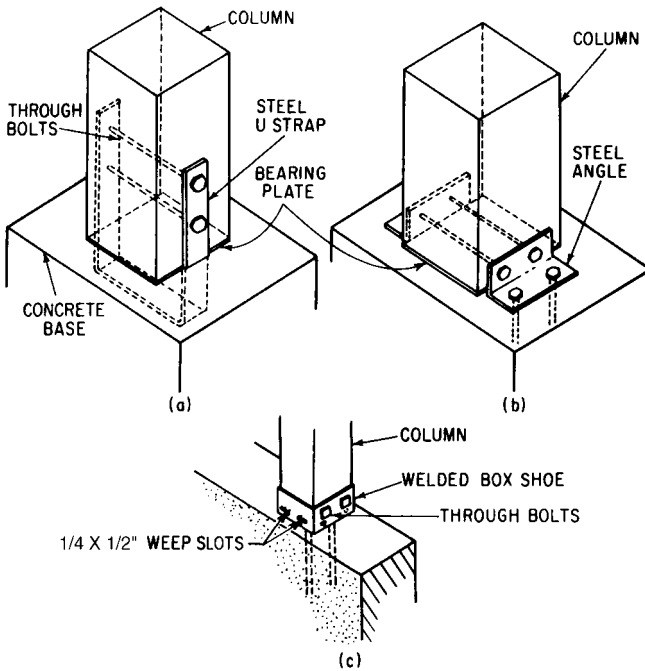


FIGURE 10.16 Typical anchorages of wood column to base: (a) column anchored to concrete base with U strap; (b) anchorage with steel angles; (c) anchorage with a welded box shoe.

tural components. Development of preengineered, prefabricated, lightweight wood and wood-steel composite trusses, however, significantly altered previous practices, which employed relatively complex and labor-intensive types of trusses, each truss requiring a detailed engineering analysis. As a result, repetitive designs are common and trusses are mass-produced in truss assembly plants.

10.22.1 Lightweight Truss Systems

Metal connector plates as a joint connection device for lumber trusses and the capability of mass producing wood trusses make lightweight, preengineered, preassembled wood trusses practical. These typically are installed at relatively close spacings (12 to 24 in c to c) and take advantage of repetitive-member design. Wider spacings (32 in c to c or more) are also used, but these are designed without consideration of repetitive-member action.

The trusses generally use dimension lumber, either visually graded or machine stress rated, for the chords and web members. A connection at a truss joint utilizes two metal connector plates, which are physically pressed into the wood on opposite faces of the joint. The plates are sized to cover all the members at the joint with a sufficient plate area to transfer the load effectively from one member to the others. The load-transfer capacity is based on an allowable load per square inch of plate.

Chord splices are accomplished in a similar manner by application of a plate on each side of the splice.

The ultimate and design load capacity of a plate depends on the steel gage and the number, size, and design of the steel projections (teeth) that are pressed into the wood. These characteristics are determined by the manufacturer. Consequently, metal connector plates from different plate manufacturers are generally not interchangeable for a specific truss. Most plate manufacturers, in conjunction with their licensed truss assemblers, provide computerized truss design support for their products. Thus, designers need show only the required truss configuration and design loading conditions on the design drawings and the truss supplier will provide all applicable truss-member designs. Virtually any truss profile can be manufactured for metal-plate connected wood trusses. More information on this type of truss can be obtained from the Truss Plate Institute (TPI) and the Wood Truss Council of America, both located in Madison, Wis.

A variation of the lightweight metal-plate connected wood truss employs much thicker metal plates in conjunction with heavier wood chord and web members, such as dimension lumber with a nominal width of 3 or 4 in or glued-laminated timbers. Use of the heavier components provides much greater load-carrying capacity. As a result, these trusses may economically be spaced much farther apart; for example, 8 ft c to c or more, and span much longer distances than the lighter trusses.

Another form of preengineered, preassembled, lightweight wood truss, commonly referred to as a composite wood-metal truss, consists of lumber chords and steel webs. One type uses metal-angle webs with nail plates formed on each end. At a joint, the teeth of the nail-plate end are pressed into a wood chord. Another configuration of composite wood-metal truss uses steel tubular web members, which are joined to the wood chords with patented steel-pin connections. The chords are typically machine stress-rated lumber or laminated veneer lumber (Art. 10.30.4). Because of the proprietary nature and uniqueness of the various types of composite wood-metal trusses, design data should be obtained from the manufacturers.

(See K. F. Faherty and T. G. Williamson, "Wood Engineering and Construction Handbook," McGraw-Hill Publishing Company, New York, for additional information.)

10.22.2 Heavy Timber Trusses

Type of heavy timber truss and arrangement of members may be chosen to suit the shape of structure, the loads, and stresses involved. The configuration most commonly used are bowstring, flat or parallel chord, pitched, triangular, and scissors (Fig. 10.17) For most commercial construction, trusses usually are spaced 8 to 24 ft apart.

Joints are critical in the design of a truss. Use of a specific truss type is often governed by joint considerations. Chords and webs may be single-leaf (or monochord), double-leaf, or multiple-leaf members. Monochord trusses and trusses with double-leaf chords and single-leaf web system are common. Web members may be attached between the chords, or web members may be in the same plane as the chords and attached with straps or gussets.

Individual truss members may be solid-sawn, glued-laminated, or mechanically laminated (rarely used in new construction). Steel rods or other steel shapes may be used as members of timber trusses if they meet design and service requirements.

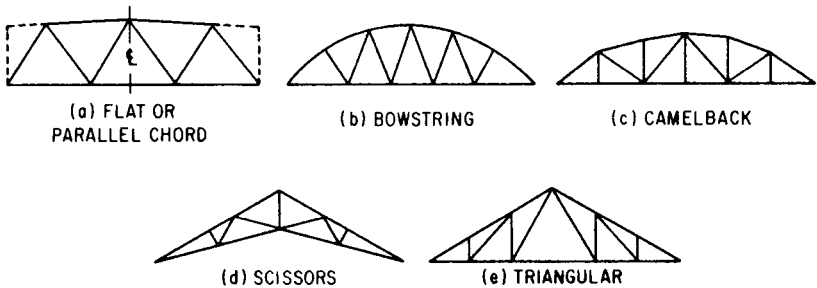


FIGURE 10.17 Types of wood trusses.

Bowstring Trusses. Spans of 100 to 200 ft are common, with single top and bottom chords of glued-laminated timber, webs of solid-sawn timber, or glulam, and steel heel plates, chord plates, and web-to-chord connections. This system is lightweight for the loads that it can carry. It can be shop or field assembled. Attention to the design of the top chord, bottom chord, and heel connections is of prime importance, since they are the major stress-carrying components. Since the top chord is nearly the shape of an ideal arch, stresses in chords are almost uniform throughout a bowstring truss and web stresses are low under uniformly distributed loads.

Parallel-Chord Trusses. Spans of 100 ft or more are possible with chords and webs of solid-sawn lumber or glulam timbers. Parallel-chord trusses are commonly used in floor systems and long-span commercial roofs. Advantages for this type of truss include ease of installation, relatively long spans, flexibility for installation of ductwork, plumbing, and electrical wiring, and overall economics.

Triangular and Scissor Trusses. Spans of 100 ft or more are possible with chords and webs of solid-sawn lumber or glulam timbers. These trusses are used extensively in residential roof construction and long-span applications in commercial roofs. Advantages are the same as those described above for parallel-chord trusses.

Truss Joints. Connectors used in truss joints include bolts, split rings, timber rivets, shear plates, lag screws, and metal connector plates, as described in Art. 10.22.1. The type of connector used depends on the type and size of wood truss components, loads to be transferred, fabrication capabilities, and aesthetics. Lag screws, bolts, timber rivets, and shear plates are suitable for field fabrication, whereas metal connector plates are almost always installed in a truss assembly plant.

Framing between Trusses. Longitudinal sway bracing perpendicular to the plane of the truss is usually provided by solid-sawn X bracing. Lateral wind bracing may be provided by end walls or intermediate walls, or both. The roof system and horizontal bracing should be capable of transferring the wind load to the walls. Knee braces between trusses and columns are often used to provide resistance to lateral loads.

Horizontal framing between trusses may consist of struts between trusses at bottom-chord level and diagonal tie rods, often of steel with turnbuckles for adjustment.

(See K. F. Faherty and T. G. Williamson, "Wood Engineering and Construction Handbook," McGraw-Hill Publishing Company, New York, for additional information.)

10.23 DESIGN OF TIMBER ARCHES

Arches typically are made of glued-laminated timber and may be two-hinged, with hinges at each base, or three-hinged with a hinge at the crown. Figure 10.18 illustrates typical forms of arches.

Tudor arches are the most commonly used type of timber arch. They are gabled rigid frames with curved haunches. The half spans on each side of the crown usually are one piece of glued-laminated timber. This type of arch is frequently used in church and commercial construction.

A-frame arches are generally used where steep pitches are required. They may spring from grade, or concrete abutments, or other suitably designed supports.

Radial arches are often used where long clear spans are required. They have been employed for clear spans up to 300 ft.

Gothic, parabolic, and three-centered arches are often selected for architectural and aesthetic considerations.

Timber arches may be tied or buttressed. If an arch is tied, the tie rods, which resist the horizontal thrust, may be above the ceiling or below grade, and simple connections may be used where the arch is supported on masonry walls, concrete piers, or columns (Fig. 10.19)

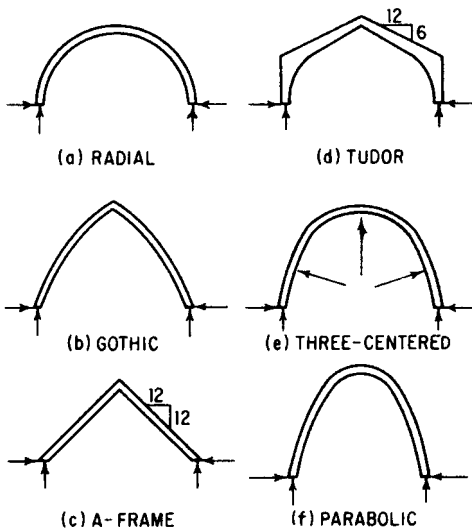


FIGURE 10.18 Types of wood arches.

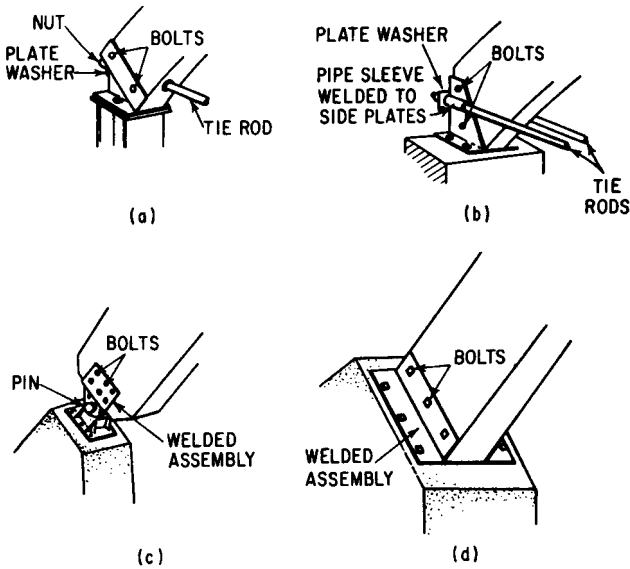


FIGURE 10.19 Bases for segmented wood arches: (a) and (b) tie rod anchored to arch shoe; (c) hinge anchorage for large arch; (d) welded arch shoe.

Arches are economical because of the ease of fabricating them and simplicity of field erection. Field splice joints are minimized; generally there is only one simple connection, at the crown (Fig. 10.20). Except for extremely long spans, they are shipped in only two pieces. Erected, they need not be concealed by false ceilings, as may be necessary with trusses. Inasmuch as arches have large cross sections, they are classified as heavy-timber construction.

Changes in moisture content may be of great importance in three-hinged arches that become horizontal, or nearly so, at the crest of the roof. Shrinkage, increasing the relative end rotations may cause a depression at the crest and create drainage problems. For such arches, therefore, consideration must be given to moisture content of the member at the time of fabrication and in service, and to the change in end angles that results from change in moisture content and shrinkage across the grain.

A long-span arch may require a splice or moment connection to segment the arch to facilitate transportation to the job site. Figure 10.21 shows typical moment connections for wood arches. Moment connections should be located at the point of minimum moment when possible.

(See K. F. Faherty and T. G. Williamson, "Wood Engineering and Construction Handbook," McGraw-Hill Publishing Company, New York, for additional information.)

10.24 TIMBER DECKING

Wood decking used for floor and roof construction may be sawn lumber with nominal thickness of 2, 3, or 4 in or glued-laminated.

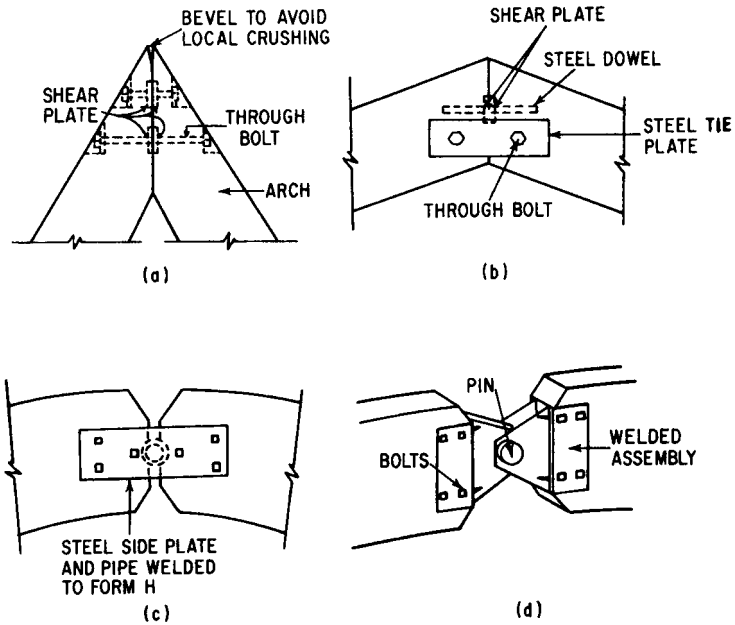


FIGURE 10.20 Crown connections for arches: (a) for arches with slope of 4:12 or greater, the connection consists of pairs of back-to-back shear plates with through bolts or threaded rods counterbored into the arch; (b) for arches with flatter slopes, shear plates centered on a dowel may be used in conjunction with tie plates and through bolts; (c) and (d) hinge at crown.

For glued-laminated decking, two or more pieces of lumber are laminated into a single decking member, usually with nominal thickness of 2, 3, or 4 in. Other thicknesses may also be available. There are no consensus standards for glued-laminated decking. Decking manufacturers should be consulted for design information.

Solid-sawn decking usually is fabricated with edges tongued and grooved, ship-lap, or groove cut for splines, to provide transfer of vertical load between pieces. The decking may be end-matched, square end, or end-grooved for splines. As indicated in Fig. 10.22, the decking may be arranged in various patterns over supports.

In Type 1, the pieces are simply supported. Type 2 has a controlled random layout. Type 3 contains intermixed cantilevers. Type 4 consists of a combination of simple-span and two-span continuous pieces. Type 5 is two-span continuous.

In Types 1, 4, and 5, end joints bear on supports. For this reason, these types are recommended for thin decking, such as 2-in.

Type 3, with intermixed cantilevers, and Type 2, with controlled random layout, are used for deck continuous over three or more spans. These types permit some of the end joints to be located between supports. Hence, provision must be made for load transfer at those joints. Tongue-and-groove edges, wood splines on each edge of the course, horizontal spikes between courses, and end matching or metal end splines may be used to transfer shear and bending stresses.

In Type 2, the distance between end joints in adjacent courses should be at least 2 ft for 2-in deck and 4 ft for 3- and 4-in deck. Joints approximately lined up

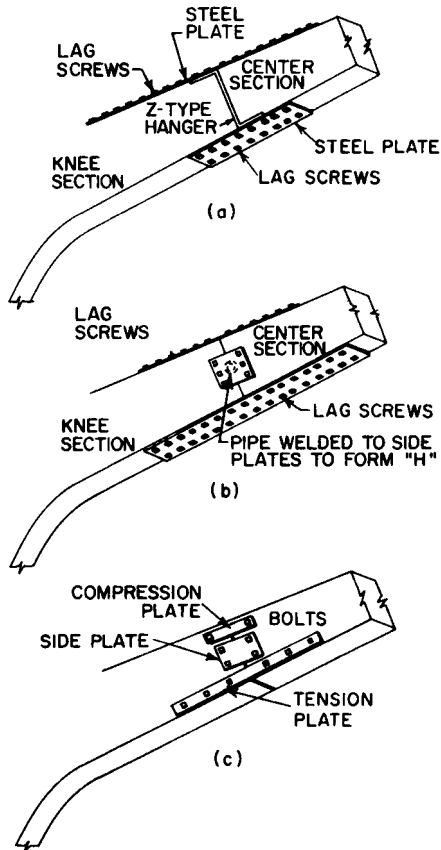


FIGURE 10.21 Schematics of some moment connections for timber arches: (a) and (b) connections with top and bottom steel plates; (c) connection with side plates.

(within 6 in of being in line) should be separated by at least two courses. All pieces should rest on at least one support, and not more than one end joint should fall between supports in each course.

In Type 3, every third course is simple span. Pieces in other courses cantilever over supports and end joints fall at alternate quarter or third points of the spans. Each piece rests on at least one support.

To restrain laterally supporting members of 2-in deck in Types 2 and 3, the pieces in the first and second courses and in every seventh course should bear on at least two supports. End joints in the first course should not occur on the same supports as end joints in the second course unless some construction, such as plywood overlayment, provides continuity. Nail end distance should be sufficient to develop the lateral nail strength required.

Heavy-timber decking is laid with wide faces bearing on the supports. Each piece must be nailed to each support. For 2-in decking a 3½-in (16d) toe and face nail should be used in each 6-in-wide piece at supports, with three nails for wider

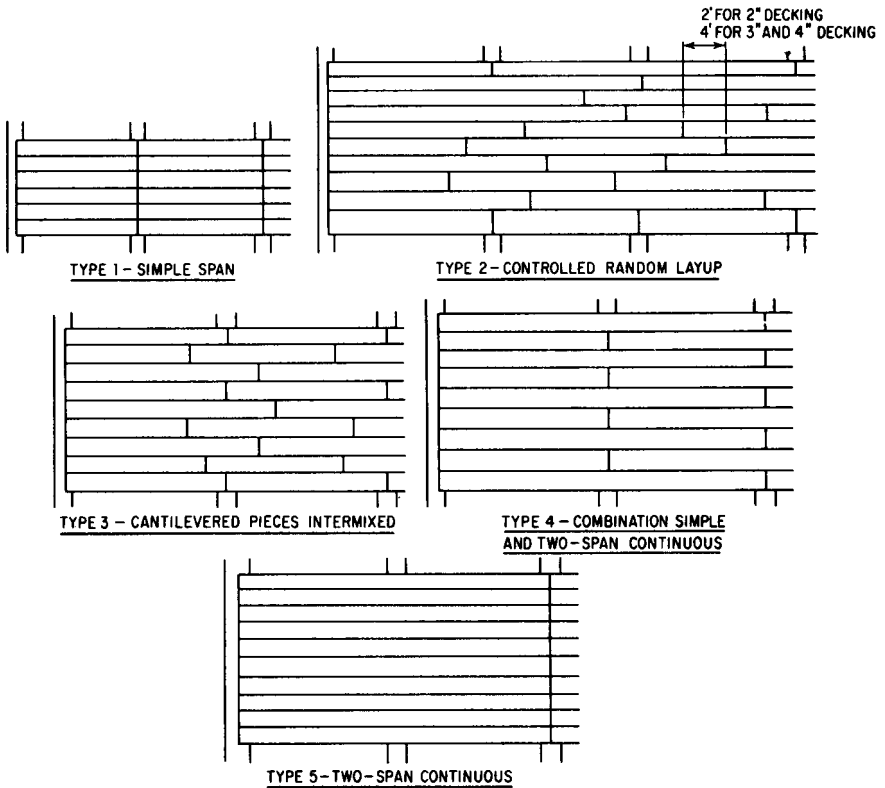


FIGURE 10.22 Typical arrangement patterns for heavy-timber decking.

pieces. Tongue-and-groove decking generally is also toenailed through the tongue. For 3-in decking, each piece should be toenailed with one 4-in (20d) spike and face-nailed with one 5-in (40d) spike at each support. For 4-in decking, each piece should be toenailed at each support with one 5-in (40d) nail and face-nailed there with one 6-in (60d) spike.

Courses of 3- and 4-in double tongue-and-groove decking should be spiked to each other with 8½-in spikes not more than 30 in apart. One spike should not be more than 10 in from each end of each piece. The spikes should be driven through predrilled holes. Two-inch decking is not fastened together horizontally with spikes.

Deck design usually is governed by maximum permissible deflection in end spans. But each design should be checked for bending stress. For load-span tables and more information on sawn lumber decking, refer to AITC 112, Standard for Heavy Timber Roof Decking, and AITC 118, Standard for 2-in Nominal Thickness Lumber Roof Decking for Structural Applications, American Institute of Timber Construction, Englewood, Colo.

10.25 WOOD-FRAME CONSTRUCTION

This is the predominant method for building single- and multifamily dwellings. Wood-frame buildings can be erected speedily and are economical to build. For

wood framing, the walls are conventionally built with studs spaced 16 or 24 in c to c. Similarly, joists and rafters, which are supported on the walls and partitions, are also usually spaced 16 or 24 in c to c. Facings, such as panel sheathing, wall-board, decking, floor underlayment, and roof sheathing, are generally available in appropriate sizes for attachment to studs, joists, and rafters with these spacings.

Wood studs are usually set in walls and partitions with wide faces perpendicular to the face of the wall or partition. The studs are nailed at the bottom to bear on a horizontal member, called the bottom or sole plate. At the top, they are nailed to a pair of horizontal members, called the top plate. These plates often are the same size as the studs. Joists or rafters may be supported on the top plate or on a header.

Studs are braced against racking by diagonal or horizontal blocking and facing materials, such as structural wood panels or in some instances by gypsum sheathing.

Two types of wood-frame construction are generally used: platform frame and plank-and-beam frame.

10.25.1 Platform Frame

In this type of construction, first-floor joists are completely covered with subflooring to form a platform on which exterior walls and interior partitions are built (Fig. 10.23). This is the type of framing usually used for single-family dwellings. (See “Manual for Wood Frame Construction,” WCD #1, American Forest & Paper Association, Washington, D.C.)

10.25.2 Plank-and-Beam Frame

In contrast to conventional framing, which utilizes joists, rafters, and studs spaced 12 to 24 in c to c, plank-and-beam frames require fewer but larger-size piers, and wood components are spaced farther apart (Fig. 10.24). In plank-and-beam framing, subfloors or roofs, typically composed of members with a nominal thickness of 2 in, are supported on beams spaced 8 ft c to c. Ends of the beams are supported on posts or concrete piers. Supplemental framing is used between posts for attachment of exterior and interior wall framing and finishes. It also provides lateral support or bracing for the structure.

Plank-and-beam framing has several cost-saving aesthetic advantages:

- Distinctive architectural effect provided by the exposed wood deck ceiling. In many cases, no further ceiling treatment is needed, except perhaps application of a sealer, stain, or paint.
- If well-planned, construction labor savings can be achieved. Larger and fewer framing members requires less handling and fewer mechanical fasteners.
- Elimination of cross bracing, which is often required in platform and balloon framing.

(See “Plank and Beam Framing for Residential Buildings,” WCD No. 4, American Forest & Paper Association, Washington, D.C.)

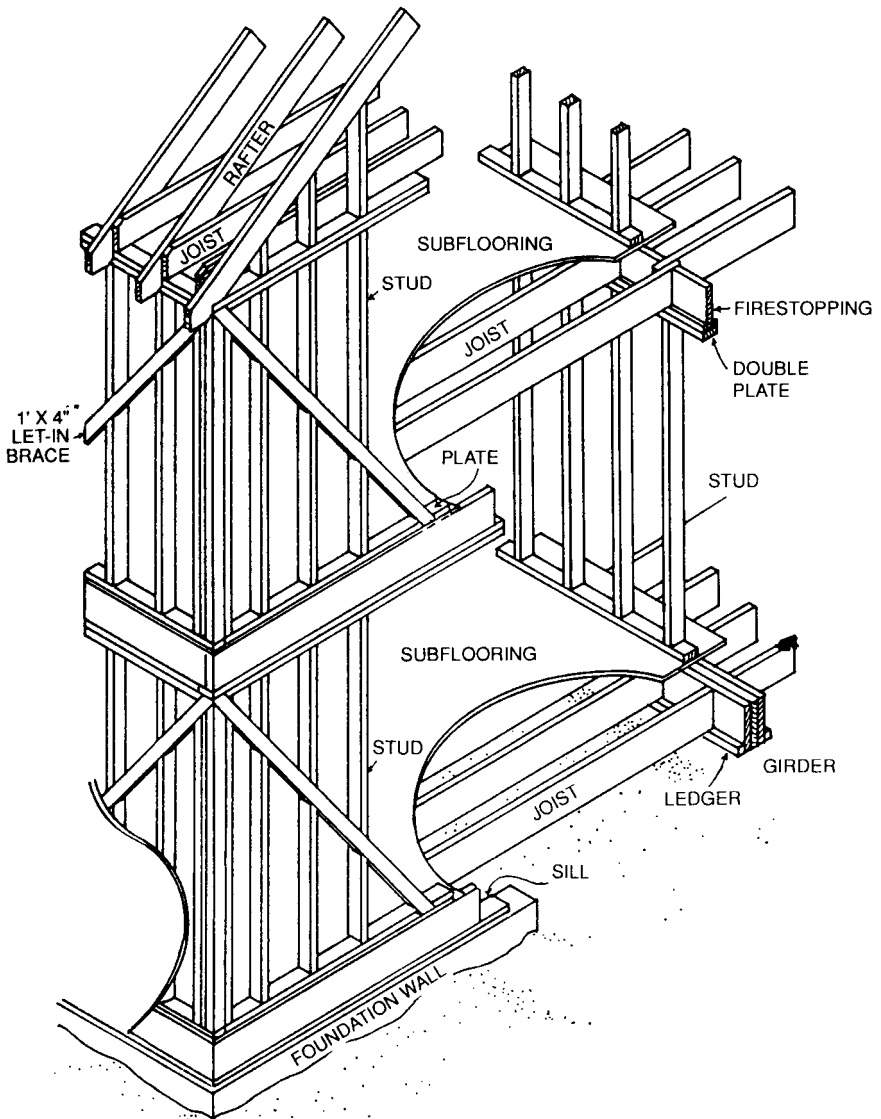


FIGURE 10.23 Platform framing for two-story building.

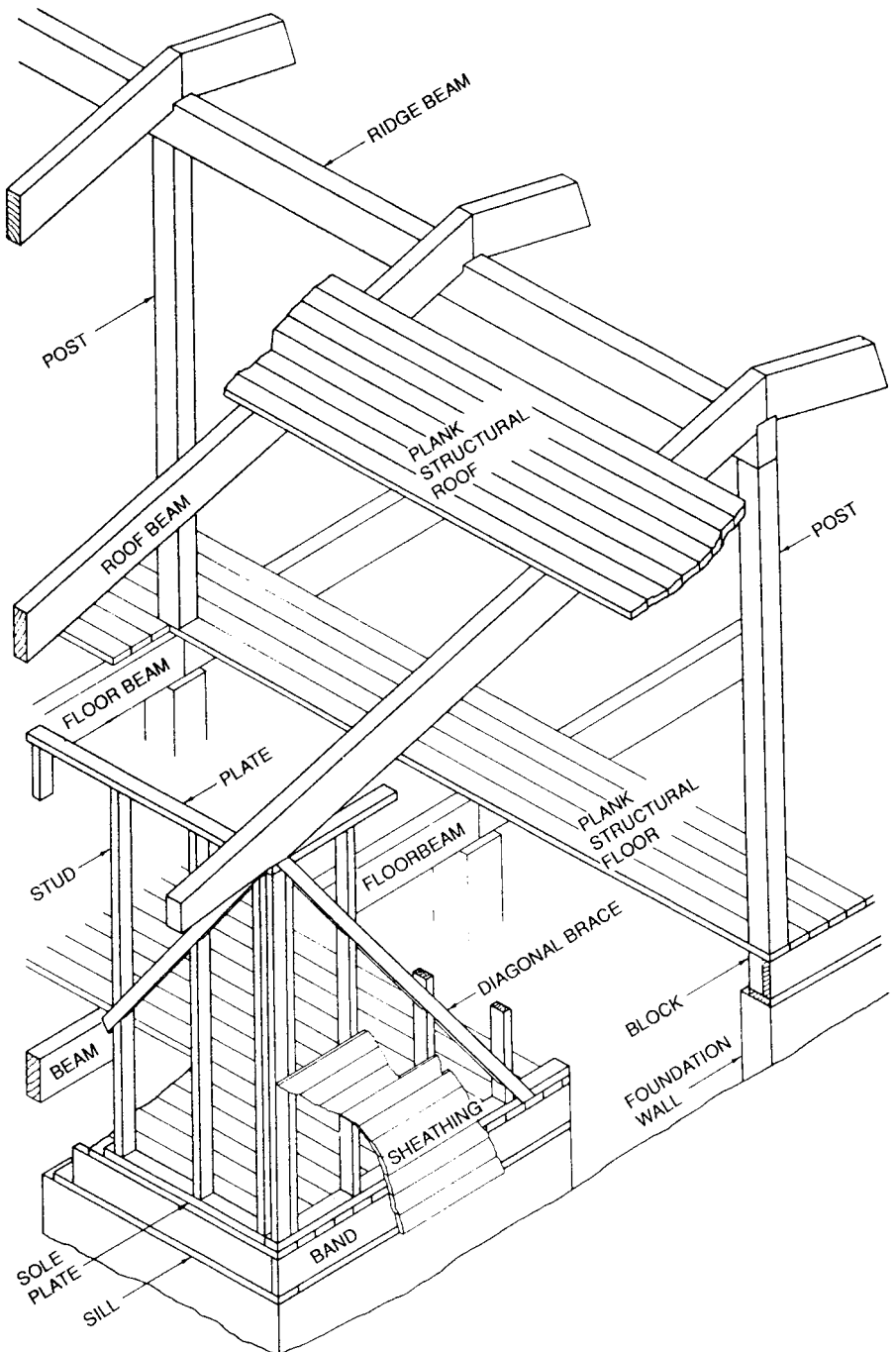


FIGURE 10.24 Plank-and-beam framing for one-story building.

10.26 PERMANENT WOOD FOUNDATIONS

Plywood and lumber walls are an alternative to concrete for foundation walls for one-story and multistory houses and other light-frame buildings. Main components of a wood foundation wall are plywood, $\frac{5}{8}$ in or more thick, and wood studs, spaced 12 in or more on centers, both pressure-treated with preservative (Fig. 10.25). Some advantages of a wood foundation over concrete are faster construction, because there is no delay due to the wait for concrete or unit masonry to cure, easier interior finishing, because wood foundations provide nailable studs for the usual finishes, the ability to erect the system in virtually any weather, and generally drier basements due to the use of gravel backfill, which facilitates drainage of water away from the foundation. Wood basements also are much warmer and more comfortable for the occupant in cold weather.

Plywood should be an exterior type or an interior type that has been bonded with exterior glue, both types manufactured to meet the requirements of U.S. Product Standard PS 1 for Construction and Industrial Plywood. Lumber should be grade marked by an approved inspection agency, should be capable of accepting pressure preservative treatment, and should be of a species for which allowable design values are given in Art. 10.3.

Treatment for the plywood and lumber involves impregnating into the wood under heat and pressure ammoniacal copper arsenate or chromated copper arsenate (A, B, or C). Salt retention should be at least 0.60 lb/ft^3 of wood, 50% more than building codes usually require for ground-contact applications. After pressure treatment, plywood should be dried to a moisture content of 18% or less, and lumber, to 19% or less. Portions of the wall more than 8 in above the ground, however, need not be pressure treated. If any of the materials have to be cut after treatment, the cut edges, unless they will be 8 in or more above grade, should be field treated with the preservatives used in the original treatment but with a minimum concen-

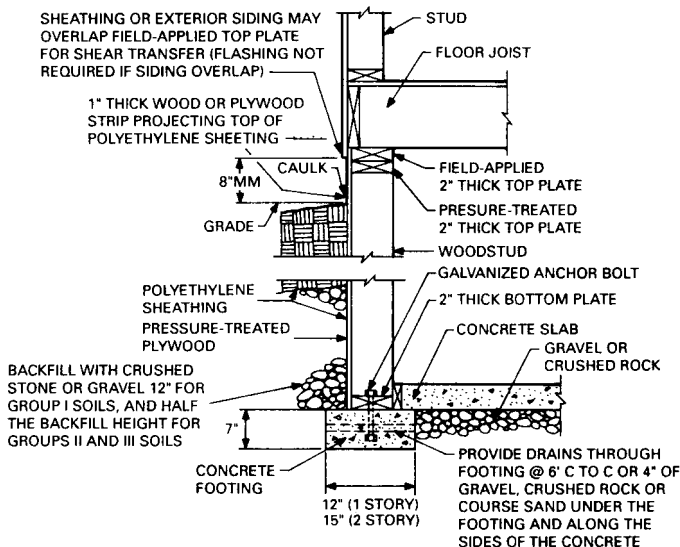


FIGURE 10.25 Wood foundation on a concrete footing.

tration of 3% in solution. To minimize end cutting and field treatment of footing plates, these members may be extended past the corners of the foundation.

Only corrosion-resistant fasteners should be used in wood foundations.

To keep the interior of the foundation walls dry, it is necessary to enclose the exterior with a waterproofing membrane, such as polyethylene film, and to ensure good drainage. For the latter purpose, the ground surface should be sloped away from the building at 1/2 in/ft for a distance of at least 6 ft and gravel should be placed against the walls and under the cellar floor.

(See "Permanent Wood Foundation System—Technical Report 7" American Forest & Paper Association; "Permanent Wood Foundations—Design and Construction Guide," Southern Forest Products Association, for additional information.)

10.27 POST FRAME AND POLE CONSTRUCTION

Wood poles and posts are used for various types of construction, including flagpoles, utility poles, and framing for buildings. These employ preservatively treated round poles or posts with square or rectangular cross sections that are set into the ground as columns. The ground furnishes vertical and horizontal support and prevents rotation at the base.

Post frame construction is used extensively in agricultural buildings and in many commercial and industrial building applications.

In buildings with post frame and pole construction, a bracing system is often provided at the top of the poles or posts to reduce bending moments at the base and to distribute loads. Design of buildings supported by poles or posts without bracing requires good knowledge of soil conditions, to eliminate excessive deflection or sidesway. For allowable foundation and lateral pressures, see the applicable building code.

TABLE 10.39 Stress Values for Treated Wood Poles*

Species	Modulus of rupture, extreme fiber in bending, psi†	Extreme fiber in bending F_b , psi‡	Modulus of elasticity E , psi‡	Compression parallel to grain F_c , psi‡
Cedar, northern white	4000	1540	600,000	740
Cedar, western red	6000	1850	900,000	1030
Douglas fir	8000	2700	1,500,000	1360
Hemlock, western	7400	2380	1,300,000	1250
Larch, western	8400	2940	1,500,000	1500
Pine, jack	6600	2100	1,100,000	1100
Pine, lodgepole	6600	1820	1,100,000	980
Pine, ponderosa	6000	1710	1,000,000	920
Pine, red or Norway	6600	2100	1,300,000	1020
Pine, southern	8000	2740	1,500,000	1360

* Air-dried prior to treatment.

† Based on *American National Standard Specifications and Dimensions for Wood Poles*, ANSI 05.1-1992.

‡ Based on ASTM D 2899-95, "Tentative Method for Establishing Design Stresses for Round Timber Piles."

Bearing values under the base of the vertical load-carrying elements should be checked. For backfilling the holes, well-tamped native soil, sand, or gravel may be satisfactory, but concrete or soil cement is more effective. Concrete fill reduces the required embedment depth and improves bearing capacity by increasing the skin-friction area of the pole.

Concrete footings used to increase bearing capacity under the base of posts should be designed to withstand the punching shear of the posts and bending moments. Thickness of concrete footings should be at least 12 in. Consideration should be given to use of concrete footings even in firm soils, such as hard dry clay, coarse firm sand, or gravel.

Methods for calculating required depth of embedment of poles and posts is provided in "Post and Pole Foundation Design," ASAE Engineering Practice, EP 486, American Society of Agricultural Engineers, St. Joseph, Mich.

Table 10.39 gives allowable stresses for treated poles. Table 10.40 lists standard dimensions for Douglas fir and southern pine poles, and Table 10.41 gives safe concentric column loads.

TABLE 10.40 Dimensions of Douglas-Fir and Southern-Pine Poles*

Class		1	2	3	4	5	6	7	9	10
Min circumference at top, in		27	25	23	21	19	17	15	15	12
Length of pole, ft	Ground-line distance from butt, † ft	Min circumference at 6 ft from butt, in								
20	4	31.0	29.0	27.0	25.0	23.0	21.0	19.5	17.5	14.0
25	5	33.5	31.5	29.5	27.5	25.5	23.0	21.5	19.5	15.0
30	5½	36.5	34.0	32.0	29.5	27.5	25.0	23.5	20.5	
35	6	39.0	36.5	34.0	31.5	29.0	27.0	25.0		
40	6	41.0	38.5	36.0	33.5	31.0	28.5	26.5		
45	6½	43.0	40.5	37.5	36.5	32.5	30.0	28.0		
50	7	45.0	42.0	39.0	38.0	34.0	31.5	29.0		
55	7½	46.5	43.5	40.5	39.0	35.0	32.5			
60	8	48.0	45.0	42.0	40.5	36.0	33.5			
65	8½	49.5	46.5	43.5	41.5	37.5				
70	9	51.0	48.0	45.0	43.0	38.5				
75	9½	52.5	49.0	46.0	44.0					
80	10	54.0	50.5	47.0						
85	10½	55.0	51.5	48.0						
90	11	56.0	53.0	49.0						
95	11	57.0	54.0	50.0						
100	11	58.5	55.0	51.0						
105	12	59.5	56.0	52.0						
110	12	60.5	57.0	53.0						
115	12	61.5	58.0							
120	12	62.5	59.0							
125	12	63.5	59.5							

* See the latest edition of Standard Specifications and Dimensions for Wood Poles, ANSI 05.1.

† The figures in this column are intended for use only when a definition of ground line is necessary to apply requirements relating to scars, straightness, etc.

TABLE 10.41 Safe Concentric Column Loads on Douglas-Fir and Southern-Pine Poles, lb*

Top diam, in	8	7	6	5
Pole class	2	3	5	6
Unsupported pole length, ft (above ground line):				
0	68,500	52,500	38,500	26,000
10	68,500	51,000	28,500	14,000
12	61,500	36,500	20,500	10,000
14	46,000	27,500	15,500	8,000
16	36,000	22,000	12,500	6,500
18	29,500	17,500	10,000	5,000
20	24,500	15,000	8,500	4,500
25	16,500	10,000	6,000	
30	12,500	7,500		
35	10,000	6,000		

* See the latest edition of Standard Specifications and Dimensions for Wood Poles, ANSI 05.1.

(See also "Design Properties of Round, Sawn and Laminated Preservatively Treated Construction Poles and Posts," ASAE Engineering Practice, EP 388.2 for additional information.)

10.28 DESIGN FOR FIRE SAFETY

For buildings with structural lumber or timber framing, fire protection of the occupants and of the property itself can be enhanced by taking advantage of the fire-endurance properties of wood in large cross sections and selecting details that make buildings fire-safe. Building materials or features alone or detection and fire-extinguishing equipment alone cannot provide maximum safety from fire in buildings. A proper combination of these measures, however, will provide the necessary fire protection (Art. 3.5).

When exposed to fire, wood forms a self-insulating surface layer of char, which provides fire protection. Even though the surface chars, the undamaged wood beneath retains most of its strength and will support loads in accordance with the capacity of the uncharred section. Heavy-timber members have often retained their structural integrity through long periods of fire exposure and remained serviceable after refinishing the charred surfaces. This fire endurance and excellent performance of heavy timber are attributable to the size of the wood members, and to the slow rate at which the charring penetrates.

The structural framing of a building, which is the criterion for classifying a building as combustible or noncombustible, has little to do with the hazard from fire to the building occupants. Most fires start in the building furnishings or contents and create conditions that render the inside of the structure uninhabitable long before the structural framing becomes involved in a fire. Thus, whether the building is of a combustible or noncombustible classification has little bearing on the potential hazard to the occupants. However, once the fire starts in the contents, the material of which the building is constructed can be of significant help in facilitating

evacuation, fire fighting, and property protection. The most important protection factors for occupants, fire fighters, and the property, as well as adjacent exposed property, are prompt detection of the fire, immediate alarm, and rapid extinguishment of the fire.

With member size of particular importance of fire endurance of wood, building codes classify buildings with wood framing as heavy-timber construction, ordinary construction, or wood-frame construction.

Heavy-timber construction is the type in which fire resistance is attained by placing requirements on the minimum size, thickness, or composition of all load-carrying wood members; by avoidance of concealed spaces under floors and roofs; by use of approved connections, construction details, and adhesives; and by providing the required degree of fire resistance in exterior and interior walls. (For design procedures and typical connector details, see "Design for Code Acceptance #2, Design of Fire-Resistive Exposed Wood Members," American Forest & Paper Association, Washington, D.C.)

Ordinary construction has exterior masonry walls and wood-framing members of sizes smaller than heavy-timber sizes.

Wood-frame construction has wood-framed walls and structural framing of sizes smaller than heavy-timber sizes.

Depending on the occupancy of a building or hazard of operations within it, a building of frame or ordinary construction may have its members covered with fire-resistive coverings. The fire endurance rating of protected wood-frame assemblies is measured in a standard fire endurance test, ASTM E119. Fire endurance ratings for various assembly designs can be found in listings that are published by recognized fire testing laboratories. These listings are updated on a regular basis. Alternatively, for many assemblies, a conservative estimate of the fire endurance rating can be obtained through a simplified calculation method ("Design for Code Acceptance #4, Component Additive Method (CAM) for Calculating and Demonstrating Assembly Fire Endurance," American Forest & Paper Association.) This method estimates the overall fire endurance rating as the sum of fire endurance times assigned to individual components of an assembly.

The interior finish on exposed surfaces of rooms, corridors, and stairways is important from the standpoint of its tendency to ignite and spread flame from one location to another. The fact that wood is combustible does not mean that it will spread flame at a hazardous rate. Code requirements to limit the flame-spread propensity of interior finishes are based on the flame-spread index (FSI) measured in the ASTM E84 tunnel test. Most wood products of sufficient thickness ($\frac{1}{4}$ in or greater) have an FSI between 76 and 200 (Class III or C). Some wood products have an FSI between 26 and 75 (Class II or B). Fire-retardant treatment can improve performance to an FSI of 25 or less (Class I or A). For an extensive list of flame spread indices of wood products, see "Design for Code Acceptance #1, Flame Spread Performance of Wood Products," American Forest & Paper Association. In general, codes require Class I or A materials for enclosed vertical exits. Class II or B materials in exit access corridors, and Class III or C materials in other areas and rooms. Thus, wood products are accepted by the codes for a wide range of interior finish uses. Most codes exclude the exposed wood surfaces of heavy-timber structural members from flame-spread requirements, if the exposed area is sufficiently small.

Fire-retardant chemicals may be impregnated in wood with recommended retentions to lower the rate of surface flame spread. After proper surface preparation, the surface is paintable. Such treatments are accepted under several specifications, including federal and military. They are recommended only for interior or dry-use service conditions or locations protected against leaching. These treatments are

sometimes used to meet a specific flame-spread rating for interior finish, or as an alternative to code requirements for noncombustible construction.

10.29 TIMBER FABRICATION AND ERECTION

During fabrication and erection processes for wood construction, wood products should be handled and covered to prevent marring of the surfaces and moisture absorption. Overstressing of members and joints during handling and erection should be avoided. Competent inspectors should check materials and workmanship.

10.29.1 Fabrication of Structural Timber

Fabrication consists of boring, cutting, sawing, trimming, dapping, routing, planing, and otherwise shaping, framing, and finishing wood units, sawn or laminated, including panels, to fit them for particular places in a final structure. Whether fabrication is performed in shop or field, the product must exhibit a high quality of workmanship.

Jigs, patterns, templates, stops, or other suitable means should be used for all complicated and multiple assemblies to ensure accuracy, uniformity, and control of all dimensions. All tolerances in cutting, drilling, and framing must comply with good practice and applicable specifications and controls. At the time of fabrication, tolerances must not exceed those listed below, unless they are not critical and not required for proper performance. Specific jobs, however, may require closer tolerances.

Location of Fastenings. Spacing and location of all fastenings within a joint should be in accordance with the shop drawings and specifications, with a maximum permissible tolerance of $\pm 1/16$ in. The fabrication of members assembled at any joint should be such that the fastenings are properly fitted.

Bolt-Hole Sizes. Bolt holes in all fabricated structural timber, when loaded as a structural joint, should be $1/16$ in larger in diameter than bolt diameter for $1/2$ -in and larger-diameter bolts, and $1/32$ in larger for smaller-diameter bolts. Larger clearances may be required for other bolts, such as anchor bolts and tension rods.

Holes and Grooves. Holes for stress-carrying bolts, connector grooves, and connector daps must be smooth and true within $1/16$ in per 12 in of depth. The width of a split-ring connector groove should be within $+0.02$ in of and not less than the thickness of the corresponding cross section of the ring. The shape of ring grooves must conform generally to the cross-sectional shape of the ring. Departure from these requirements may be allowed when supported by test data. Drills and other cutting tools should be set to conform to the size, shape, and depth of holes, grooves, daps, etc., specified in the "National Design Specification for Wood Construction," American Forest & Paper Association.

Lengths. Members should be cut within $\pm 1/16$ in of the indicated dimension when they are up to 20 ft long, and $\pm 1/16$ in per 20 ft of specified length when they are over 20 ft long. Where length dimensions are not specified or critical, these tolerances may be waived.

End Cuts. Unless otherwise specified, all trimmed square ends should be square within $\frac{1}{16}$ in per foot of depth and width. Square or sloped ends to be loaded in compression should be cut to provide contact over substantially the complete surface.

10.29.2 Timber Erection

Erection of timber framing requires experienced crews and adequate lifting equipment to protect life and property and to assure that the framing is properly assembled and not damaged during handling.

Each shipment of timber should be checked for tally and evidence of damage. Before erection starts, plan dimensions should be verified in the field. The accuracy and adequacy of abutments, foundations, piers, and anchor bolts should be determined. The erector must see that all supports and anchors are complete, accessible, and free from obstructions.

Job-Site Storage. If wood members must be stored at the site, they should be placed where they do not create a hazard to other trades or to the members themselves. All framing stored at the site should be set above the ground on appropriate blocking. Where practical, the members or bundles of material should be separated with strips, so that air may circulate around all sides. The top and all sides of each storage pile should be covered with a moisture-resistant covering that provides protection from the elements, dirt, and job-site debris. The use of clear polyethylene films is not recommended, since wood members may be bleached by sunlight. Individual wrappings should be slit or punctured on the lower side to permit drainage of water that accumulates inside the wrapping. Particular care should be taken with members such as glued-laminated timber that may be exposed to view in the completed structure.

Glued-laminated members of Premium and Architectural Appearance (and Industrial Appearance in some cases) are usually shipped with a protective wrapping of water-resistant paper. While this paper does not provide complete freedom from contact with water, experience has shown that protective wrapping is necessary to ensure proper appearance after erection. Though used specifically for protection in transit, the paper should remain intact during job-site storage. Removal of the paper during erection is a designer or contractor option. For example, if the paper is removed from isolated areas to make connections from one member to another, either the paper should be replaced and should remain in position until all the wrapping is removed or all the paper should be removed to minimize uneven discoloration due to sun bleaching.

At the site, to prevent surface marring and damage to wood members, the following precautions should be taken:

Lift members or roll them on dollies or rollers out of railroad cars. Unload trucks by hand, forklift, or crane. Do not dump, drag, or drop members.

During unloading with lifting equipment, use fabric belts, or other slings that will not mar the wood. Provide additional protective blocking or padding at edges of members.

Guard against soiling, dirt, footprints, abrasions, or injury to shaped edges or sharp corners.

Equipment. Adequate equipment of proper load-handling capacity, with control for movement and placing of members, should be used for all operations. It should be of such nature as to ensure safe and expedient placement of the material. Cranes and other mechanical-devices must have sufficient controls that beams, columns, arches, or other elements can be eased into position with precision. Slings, ropes, and other securing devices must not damage the materials being placed.

The erector should determine the weights and balance points of the framing members before lifting begins, so that proper equipment and lifting methods may be employed. When long-span timber trusses are raised from a flat to a vertical position preparatory to lifting, stresses entirely different from design stresses may be introduced. The magnitude and distribution of these stresses depend on such factors as weight, dimensions, and type of truss. A competent rigger will consider these factors in determining how much suspension and stiffening, if any, is required and where it should be located.

Accessibility. Adequate space should be available at the site for temporary storage of materials from time of delivery to the site to time of erection. Material-handling equipment should have an unobstructed path from job-site storage to point of erection. Whether erection must proceed from inside the building area or can be done from outside will determine the location of the area required for operation of the equipment. Other trades should leave the erection area clear until all members are in place and are either properly braced by temporary bracing or are permanently braced in the building system.

Assembly and Subassembly. Whether done in a shop or on the ground or in the air in the field, assembly and subassembly are dependent on the structural system and the various connections involved.

Care should be taken with match-marking on custom materials. Assembly must be in accordance with the approved shop drawings. Any additional drilling or dapping, as well as the installation of all field connections, must be done in a workmanlike manner.

Heavy-timber trusses are often shipped partly or completely disassembled. They are assembled on the ground at the site before erection. Arches, which are generally shipped in half sections, may be assembled on the ground or connections may be made after the half arches are in position. When trusses and arches are assembled on the ground at the site, assembly should be on level blocking to permit connections to be properly fitted and securely tightened without damage. End compression joints should be brought into full bearing and compression plates installed where intended.

Before erection, the assembly should be checked for prescribed overall dimensions, prescribed camber, and accuracy of anchorage connections. Erection should be planned and executed in such a way that the close fit and neat appearance of joints and the structure as a whole will not be impaired.

Field Welding. Where field welding is required, the work should be done by a qualified welder in accordance with job plans and specifications, approved shop drawings, and specifications of the American Institute of Steel Construction and the American Welding Society.

Cutting and Fitting. All connections should fit snugly in accordance with job plans and specifications and approved shop drawings. All cutting, framing, and boring should be done in accordance with good shop practices. Any field cutting,

dapping, or drilling should be done in a workmanlike manner, with due consideration given to final use and appearance.

Bracing. Structural elements should be placed to provide lateral restraint and vertical support, to ensure that the complete assembly will form a stable structure. This bracing may extend longitudinally and transversely. It may comprise sway, cross, vertical, diagonal, and like members that resist wind, earthquake, erection, acceleration, braking, and other forces. And it may consist of knee braces, cables, rods, struts, ties, shores, diaphragms, rigid frames, and other similar components in combinations.

Bracing may be temporary or permanent. Permanent bracing, required as an integral part of the completed structure, is shown on the architectural or engineering plans and usually is also referred to in the job specifications. Temporary construction bracing is required to stabilize or hold in place permanent structural elements during erection until other permanent members that will serve the purpose are fastened in place. This bracing is the responsibility of the erector, who normally furnishes and erects it. Protective corners and other protective devices should be installed to prevent members from damaged by the bracing.

In wood truss construction, temporary bracing can be used to plumb trusses during erection and hold them in place until other secondary framing and roof sheathing are installed. The major portion of temporary bracing for trusses is left in place, because it is designed to brace the complete structure against lateral forces.

Failures during erection occur occasionally and regardless of construction material used. The blame can usually be placed on insufficient or improperly located temporary erection guys or braces, overloading with construction materials, or an externally applied force sufficient to render temporary erection bracing ineffective. (See "Handling, Installing and Bracing Metal Plate Connected Wood Trusses," Truss Plate Institute, Madison, Wis., for guidance in erection of lightweight metal plate connected, wood trusses.)

Structural members of wood must be stiff, as well as strong. They must also be properly guyed or laterally braced, both during erection and permanently in the completed structure. Large rectangular cross sections of glued-laminated timber have relatively high lateral strength and resistance to torsional stresses during erection. However, the erector must never assume that a wood arch, beam, or column cannot buckle during handling or erection.

Specifications often require that:

1. Temporary bracing shall be provided to hold members in position until the structure is complete.
2. Temporary bracing shall be provided to maintain alignment and prevent displacement of all structural members until completion of all walls and decks.
3. The erector should provide adequate temporary bracing and take care not to overload any part of the structure during erection.

While the magnitude of the restraining force that should be provided by a cable guy or brace cannot be precisely determined, general experience indicates that a brace is adequate if it supplies a restraining force equal to 2% of the applied load on a column or of the force in the compression flange of a beam. It does not take much force to hold a member in line; but once it gets out of alignment, the force then necessary to hold it is substantial.

10.30 ENGINEERED GLUED WOOD PRODUCTS

These products, which include glued-laminated timber, prefabricated wood I joists, and structural composite lumber (SCL), are used extensively in residential and nonresidential building construction. They are used instead of nonwood products, such as steel framing elements, or as substitutes for conventional sawn-lumber products.

10.30.1 Characteristics of Glulam Timber

Glued-laminated timber, or glulam as it is often referred to, is the oldest type of the engineered glued wood products. With the employment of wet-use adhesives for laminating, glulam elements may be used in applications where they may be exposed to the elements. These applications include exterior building components, utility structures, marinas and wharfs, and bridge structures, such as pedestrian, highway, and railway bridges.

Preservative Treatment. For any exposed application, glulam members should be pressure impregnated with an improved preservative, such as creosote, pentachlorophenol (penta) in various carriers, and waterborne arsenicals. All treatments, however, are not compatible with all species; a specific treatment may not be available in the job-site region. Some treatments may be pressure impregnated into the laminations before the member is glued, whereas other treatments can be used only in conjunction with the finished product. (See Standard C-18, American Wood Preservers Association, Granbury, TX, and technical notes on pressure treating of glulam of Engineered Wood Systems, Tacoma, WA and AITC, Englewood, CO.)

Versatility and Use. The most versatile of the engineered glued wood products, glulam can be fabricated in a wide variety of shapes, such as those shown in Fig. 10.27. Short-span glulam beams with constant rectangular cross sections, the most commonly used shape, are typically available as “stock beams” from distribution centers throughout the United States for use in residential and light commercial construction. Inventoried in a variety of sizes and lengths, stock beams are often used for headers and floorbeams as well as for other uses.

Straight or curved beams can be manufactured in lengths of over 100 ft and with large cross-sectional areas. Glulam arches have been erected to span 300 ft or more. For structures requiring very large spans, such as stadiums needing spans of 500 ft or more, glulam timber domes are often the most economical framing system and are esthetically pleasing.

The limitations on length for transporting glulam members from the manufacturing plant to the job site with available rail or truck facilities may control the size of glulam members that can be used for long spans without splicing. Designers and contractors should closely coordinate arrangements for transportation of long glulam components with the manufacturer.

10.30.2 Manufacture of Glulam Members

Structural glued-laminated timber is made by bonding layers of lumber together with adhesive so that the grain direction of all laminations is essentially parallel.

Narrow boards may be edge-glued, and short boards may be end-joined to create greater lengths. The resultant wide and long laminations are then face-glued into large, manufactured timbers. Figure 10.26 shows different types of glulam beams.

Recommended practice calls for lumber of nominal 1- and 2-in thicknesses for laminating. The lumber is dressed to $\frac{3}{4}$ -, $1\frac{3}{8}$ - and $1\frac{1}{2}$ -in thicknesses, depending on the species before gluing. The thinner laminations are generally used in curved members.

Virtually any species of wood can be used in the laminating process if the design values have been determined. Different species can be intermixed within the depth of a section to achieve optimum resource utilization. Higher-strength species are positioned in a beam in zones that will be subjected to high stresses under design loads. Lower-strength species can be placed in zones with lower in-service stresses. Similarly, manufacturer of glulam beams can be based on a graded layup concept. This requires that laminations with a higher lumber grading be used in zones subjected to high design stresses, and lower grades, in lower-stressed areas of the member. As a consequence, glulam members are a resource-efficient wood product, since varying grades and species can be used to achieve desired performance.

Constant-depth members normally are a multiple of the thickness of the lamination stock used. Variable-depth members, because of tapering or special assembly techniques, may not be exact multiples of these lamination thicknesses.

Nominal width of stock, in	4	6	8	10	12
Standard member finished width, in	3 or $3\frac{1}{8}$	5 or $5\frac{1}{8}$	$6\frac{3}{4}$	$8\frac{1}{2}$ or $8\frac{3}{4}$	$10\frac{1}{2}$ or $10\frac{3}{4}$

Standard widths as listed above are most economical, since they represent the maximum width of board normally obtained from the stock used in laminating. Other widths, such as $3\frac{1}{2}$ or $5\frac{1}{2}$ in, which fit well with conventional 2×4 and 2×6 framing, are also available in many market areas.

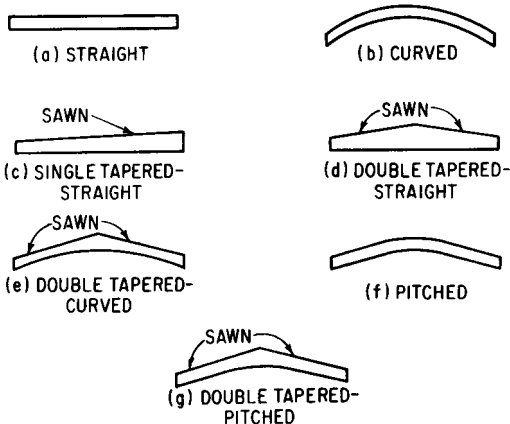


FIGURE 10.26 Types of glued-laminated beams.

When members wider than the stock available are required, laminations may consist of two boards side by side. These edge joints must be staggered, vertically in horizontally laminated beams (load acting normal to wide faces of laminations), and horizontally in vertically laminated beams (load acting normal to the edge of laminations). In horizontally laminated beams, edge joints need not be edge-glued. Edge gluing is required in vertically laminated beams.

Edge and face gluings are the simplest to make, end gluings the most difficult.

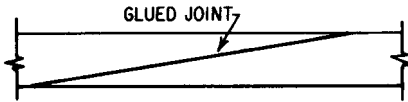


FIGURE 10.27 Plane sloping scarf.

Although no longer used in the glulam industry, a plane sloping scarf (Fig. 10.27), in which the tapered surfaces of laminations are glued together, can develop 85 to 90% of the strength of an unscarfed, clear, straight-grained control specimen. Finger joints (Fig. 10.28) are

less wasteful of lumber and hence are widely used by the glulam industry. Quality can be adequately controlled in machine cutting and in high-frequency gluing. A combination of thin tip, flat slope on the side of the individual fingers, and a narrow pitch is desired. The length of fingers should be kept short for savings of lumber, but long for maximum strength. Typical finger joint lengths are 1"–1½".

The usefulness of structural glued-laminated timbers is determined by the lumber used and glue joint produced. Certain combinations of adhesive, treatment, and

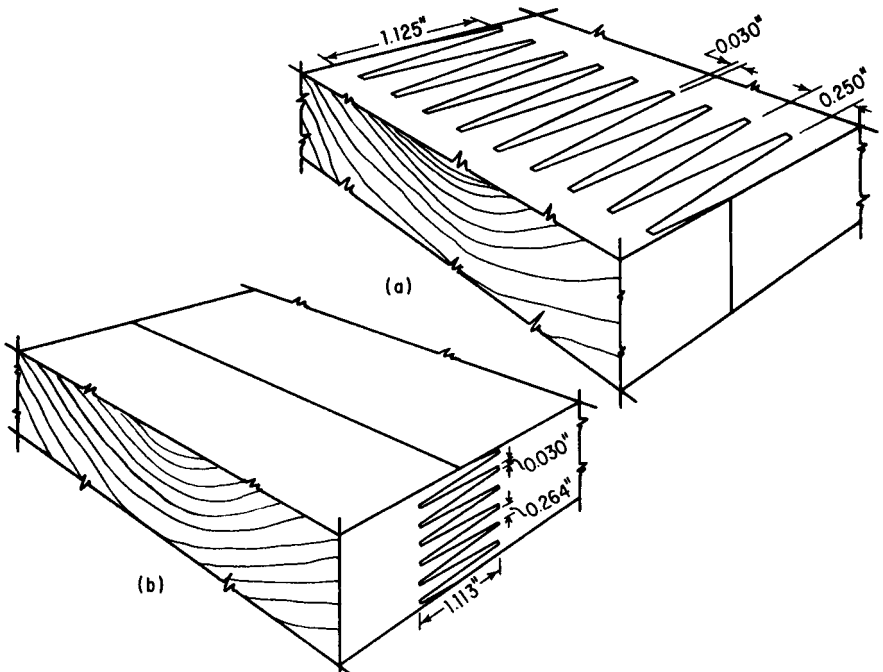


FIGURE 10.28 Finger joints: (a) fingers formed by cuts perpendicular to the wide faces of the boards; (b) fingers formed by cuts perpendicular to the edges.

wood species do not produce the same quality of glue bond as other combinations, although the same gluing procedures are used. Thus, a combination must be supported by adequate experience with a laminator's gluing procedure.

The only adhesives currently recommended for wet-use and preservative-treated lumber, whether gluing is done before or after treatment, are resorcinol and phenol-resorcinol resins. Melamine and melamine-urea blends are also used for high-frequency curing of end joints.

Glued joints may be cured with heat by several methods. Radio-frequency (RF) curing of glue lines is used for end joints and for limited-size members where there are repetitive gluings of the same cross section. Low-voltage resistance heating, where current is passed through a strip of metal to raise the temperature of a glue line, formerly was used for attaching thin facing pieces. The metal could be left in the glue line as an integral part of the completed member. Printed electric circuits, in conjunction with adhesive films, and adhesive films impregnated on paper or on each side of a metal conductor placed in the glue line are other alternatives.

Preheating the wood to ensure reactivity of the applied adhesive has limited application in structural laminating. The method requires adhesive application as a wet or dry film simultaneously to all laminations, and then rapid handling of multiple laminations.

Curing the adhesive at room temperature has many advantages. Since wood is an excellent insulator, a long time is required for elevated ambient temperatures to reach inner glue lines of a large assembly. With room-temperature curing, equipment needed to heat the glue line is not required, and the possibility of injury to the wood from high temperature is avoided.

10.30.3 Prefabricated Wood I Joists

Flanges of prefabricated wood I-shaped joists are either sawn lumber, visually graded or machine stress rated, or some type of structural composite lumber product, such as laminated veneer lumber (LVL) (Art. 10.30.4). The web members can be either plywood or oriented strand board (OSB) although OSB is most often used (Art. 10.12).

Numerous manufacturers produce wood I joists, and the flange and web materials used depend on manufacturer preference. Inasmuch as these joists are proprietary products, the manufacturer provides design information, usually in the form of load/span tables, as well as installation and handling guidelines.

Manufacturers establish design values for these products in accordance with the provisions of the "Specification for Establishing and Monitoring Structural Capacities of Prefabricated Wood I-Joists," ASTM D5055. Each manufacturer uses the design values thus determined to obtain National Evaluation Service (NER) building-code approval from model code sponsors and to generate proprietary load/span tables. To establish consistent performance levels for I-joists, APA—The Engineered Wood Association promulgated APA Standard PRI-400 for I-joists used in residential floor construction. This standard has been recognized by all of the model code agencies.

Wood I joists are available with a wide range of depths and load/span capabilities. Although joist sizes vary with manufacturer preference, most manufacturers produce joists with depths of 9½ in or 11⅞ in for direct substitution for 2 × 10 and 2 × 12 dimension lumber. Other depths—14 and 16 in and deeper—also are available. The deeper products are typically used in longer-span applications, such as for light commercial construction.

Prefabricated wood I joists have many advantages that make them an economical construction material. Some of these advantages are:

Manufactured product. They are shipped to the job site precut to length, thereby eliminating waste. Consistent product quality is assured by the manufacturing process so that all material arriving at the job site is usable. Wood I joists are manufactured from dry components, thus eliminating shrinkage, warping, and twisting.

Long lengths available. I joists can be manufactured and shipped to the job site in long lengths. This minimizes labor costs due to handling and allows the joists to be used in multispan applications.

Lightweight. Their low weight makes it very easy for construction workers to easily handle long lengths on the job site, whether for long clear spans or multiple spans. A typical residential I-joist only weighs 2 pounds per foot.

Ease of fabrication. The structural panel webs are easily cut to permit passage of wiring, conduit, plumbing, and mechanical ductwork. Manufacturing provide charts that indicate the maximum permissible size of round or rectangular openings that can be cut in the web without adversely affecting structural performance of the joists. APA also publishes hole charts consistent with PRI 400.

10.30.4 Structural Composite Lumber

Structural composite lumber (SCL) comprises a family of secondary manufactured engineered glued wood products. The most widely available type of SCL, laminated veneer lumber (LVL), is similar to plywood in that thin sheets of veneer are structurally bonded together to create large panels, usually in widths of 2 or 4 ft. These panels are typically produced in thicknesses of 1½ or 1¾ in, and in long lengths. (Lengths vary, depending on the manufacturer). However, unlike plywood, which has the veneers cross-banded, all veneers in LVL products are oriented with grain approximately parallel, much like the positioning of sawn lumber in glulam timber.

Another type of SCL, parallel-strand lumber (PSL), is manufactured from long strands of veneer rather than veneer sheets as used in LVL manufacturing. PSL is manufactured in a variety of widths ranging from 1¾ in up to 7 in. Various depths and lengths are also produced. Designers should consult the manufacturers for information on size available.

The large PSL sections can be resawn into a variety of smaller dimension lumber or timber products. An extensive use of this product is for beams and headers. Two pieces of 1¾-in-wide LVL can be nailed together to create a 3½-in-wide beam for use in conventional 2 × 4 framing. Wider beams of LVL can be created by nailing three or more pieces together or by cutting larger sections from PSL billets. SCL is used for scaffold plank, truss chords, flanges for wood I joists, ridge beams in mobile homes, and a myriad of other building and industrial uses.

Structural design properties for SCL are generally much higher than comparable values for sawn lumber. SCL is available with allowable bending stresses up to 3000 psi with corresponding modulus of elasticity up to 2,100,000 psi. ASTM Standard D5456 sets forth the procedures for determining the design properties for SCL products and published values vary among the various manufacturers. APA has promulgated a performance standard for LVL, PRL 501, which establishes uniform design properties among manufacturers.

In addition to exhibiting higher strength characteristics than other wood products, design properties for SCL products have less variability. This is largely due to the control in manufacturing of natural strength-reducing characteristics of wood, such as slope of grain, knots, and density. Also, the random dispersal of these strength-reducing characteristics throughout the finished member tends to offset the individual effects of these defects on the overall strength of the end product, much like the use of varying grades of sawn lumber in the manufacture of glulam timber. This combination of higher strengths and reduced variability makes SCL an economical wood structural material.

10.30.5 Environmental Considerations

With the increasing emphasis on more efficient uses of the available wood-fiber resource, engineered glued wood products are becoming more attractive. Each of the glued products described in the preceding articles makes optimum use of the base wood products in creating high-end, high-quality engineered products. Innovations in the engineered wood products industry are ongoing and it is these innovative engineered wood products that will allow wood to continue to be a viable construction material for building applications in the future.