11

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Wood Design and Construction

Wood is remarkable for its beauty, versatility, strength, durability, and workability. It possesses a high strength-to-weight ratio. It has flexibility. It performs well at low temperatures. It withstands substantial overloads for short periods. It has low electrical and thermal conductance. It resists the deteriorating action of many chemicals that are extremely corrosive to other building materials. There are few materials that cost less per pound than wood.

As a consequence of its origin, wood as a building material has inherent characteristics with which users should be familiar. For example, although cut simultaneously from trees growing side by side in a forest, two boards of the same species and size most likely do not have the same strength. The task of describing this nonhomogeneous material, with its variable biological nature, is not easy, but it can be described accurately, and much better than was possible in the past because research has provided much useful information on wood properties and behavior in structures.

Research has shown, for example, that a compression grade cannot be used, without modification, for the tension side of a deep bending member. Also, a bending grade cannot be used, unless modified, for the tension side of a deep bending member or for a tension member. Experience indicates that typical growth characteristics are more detrimental to tensile strength than to compressive strength. Furthermore, research has made possible better estimates of wood's engineering qualities. No longer is it necessary to use only visual inspection, keyed to averages, for estimating the engineering qualities of a piece of wood. With a better understanding of wood now possible, the availability of sound structural design criteria, and development of economical manufacturing processes, greater and more efficient use is being made of wood for structural purposes.

Improvements in adhesives also have contributed to the betterment of wood construction. In particular, the laminating process, employing adhesives to build up thin boards into deep timbers, improves nature. Not only are stronger structural members thus made available, but also higher grades of lumber can be placed in regions of greatest stress and lower grades in regions of lower stress, for overall economy. Despite variations in strength of wood, lumber can be transformed into glued-laminated timbers of predictable strength and with very little variability in strength.

11.1 Basic Characteristics of Wood

Wood differs in several significant ways from other building materials, mainly because of its cellular structure. Because of this structure, structural properties depend on orientation. Although 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 across the grain.) Parallel to the grain, wood possesses high strength and stiffness. Across the grain, strength is 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 for dimensional changes 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 from this cause vary in the three grain directions; size changes about 6 to 16% tangentially, 3 to 7% radially, but only 0.1 to 0.3% longitudinally.

Wood offers numerous advantages nevertheless in construction applications—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 is resistant to many chemicals that are highly corrosive to other materials. It has high shock-absorption capacity. It can withstand large overloads of short time duration. It has good wearing qualities, particularly on its end grain. It can be bent easily to sharp curvature. A wide range of finishes can be applied for decoration or protection. Wood can be used in both wet and dry applications. Preservative treatments are available for use when necessary, as are fire retardants. Also, there is a choice of a wide range of species with a wide range of properties.

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

11.1.1 Moisture Content of Wood

Wood is unlike most structural materials in regard to the causes of its dimensional changes, which are primarily from gain or loss of moisture, not change in temperature. For this reason expansion joints are seldom required for wood structures to permit movement with temperature changes. It partly accounts for the fact that wood structures can withstand extreme temperatures without collapse.

A newly felled tree is green (contains moisture). When the greater part of this water is being removed, seasoning first allows free water to leave the cavities in the wood. A point is reached where these cavities contain only air, and the cell walls still are full of moisture. The moisture content at which this occurs, the fiber-saturation point, varies from 25 to 30% of the weight of the oven-dry wood.

During removal of the free water, the wood remains constant in size and in most properties (weight decreases). Once the fiber-saturation point has been passed, shrinkage of the wood begins as the cell walls lose water. Shrinkage continues nearly linearly down to zero moisture content (Table 11.1). (There are, however, complicating factors, such as the effects of timber size and relative rates of moisture movement in three directions: longitudinal, radial, and tangential to the growth rings.) Eventually, the wood assumes a condition of equilibrium, with the final moisture content dependent on the relative humidity and temperature of the ambient air. Wood swells when it absorbs moisture, up to the fiber-saturation point. The relationship of wood moisture content, temperature, and relative humidity can actually define an environment (Fig. 11.1).

This explanation has been simplified. Outdoors, rain, frost, wind, and sun can act directly on the wood. Within buildings, poor environmental conditions may be created for wood by localized heating, cooling, or ventilation. The conditions of service must be sufficiently well known to be specifiable. Then, the proper design value can be assigned to wood and the most suitable adhesive selected.

Dry Condition of Use ▪ Design values for dry conditions of use are applicable for normal loading when the wood moisture content in service is less than 16%, as in most covered structures.

Dry-use adhesives perform satisfactorily when the moisture content of wood does not exceed 16% for repeated or prolonged periods of service and are to be used only when these conditions exist.

Table 11.1 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, %
Softwoods:‡									
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
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
Gum, 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	8.9	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 weight of oven-dry wood. These shrinkage values have been taken as one-third the shrinkage to the oven-dry conditions as given in the last three columns.

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

‡ 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 dimension.

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

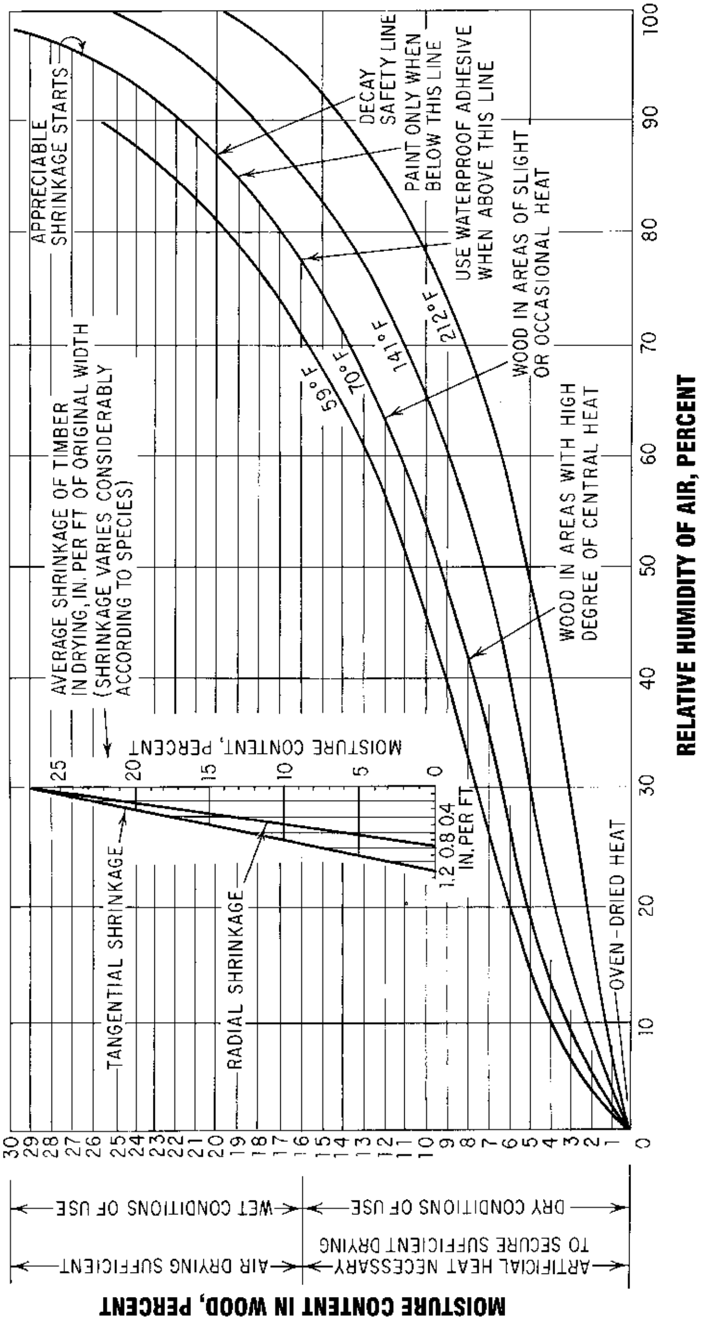

Fig. 11.1 Chart shows the approximate relationship for wood of equilibrium moisture content, temperature, and relative humidity. The triangular diagram indicates the effect of wood moisture content on the shrinkage of wood.

Wet Condition of Use ▪ Design values for wet condition of use are applicable for normal loading when the moisture content in service is 16% or more. This may occur in members not covered or in covered locations of high relative humidity.

Wet-use adhesives will perform satisfactorily for all conditions, including exposure to weather, marine use, and where pressure treatments are used, whether before or after gluing. Such adhesives are required when the moisture content exceeds 16% for repeated or prolonged periods of service.

11.1.2 Checking in Timbers

Separation of grain, or checking, is the result of rapid lowering of surface moisture content combined with a difference in moisture content between inner and outer portions of the piece. As wood loses moisture to the surrounding atmosphere, the outer cells of the member lose at a more rapid rate than the inner cells. As the outer cells try to shrink, they are restrained by the inner portion of the member. The more rapid the drying, the greater the differential in shrinkage between outer and inner fibers and the greater the shrinkage stresses. Splits may develop. Splits are cracks from separation of wood fibers across the thickness of a member that extend parallel to the grain.

Checks, radial cracks, affect the horizontal shear strength of timber. A large reduction factor is applied to test values in establishing design values, in recognition of stress concentrations at the ends of checks. Design values for horizontal shear are adjusted for the amount of checking permissible in the various stress grades at the time of the grading. Since strength properties of wood increase with dryness, checks may enlarge with increasing dryness after shipment without appreciably reducing shear strength.

Cross-grain checks and splits that tend to run out the side of a piece, or excessive checks and splits that tend to enter connection areas, may be serious and may require servicing. Provisions for controlling the effects of checking in connection areas may be incorporated into design details.

To avoid excessive splitting between rows of bolts due to shrinkage during seasoning of solid-sawn timbers, the rows should not be spaced more than 5 in apart, or a saw kerf, terminating in a bored hole, should be provided between the lines of bolts. Whenever possible, maximum end distances for connections should be specified to minimize the effect of checks running into the joint area. Some designers require stitch bolts in members, with multiple connections loaded at an angle to the grain. Stitch bolts, kept tight, will reinforce pieces where checking is excessive.

One principal advantage of glued-laminated timber construction is relative freedom from checking. Seasoning checks may however, occur in laminated members for the same reasons that they exist in solid-sawn members. When laminated members are glued within the range of moisture contents set in American National Standard, "Structural Glued Laminated Timber," ANSI/AITC A190.1, they will approximate the moisture content in normal-use conditions, thereby minimizing checking. Moisture content of the lumber at the time of gluing is thus of great importance to the control of checking in service. However, rapid changes in moisture content of large wood sections after gluing will result in shrinkage or swelling of the wood, and during shrinking, checking may develop in both glued joints and wood.

Differentials in shrinkage rates of individual laminations tend to concentrate shrinkage stresses at or near the glue line. For this reason, when checking occurs, it is usually at or near glue lines. The presence of wood-fiber separation indicates glue bonds and not delamination.

In general, checks have very little effect on the strength of glued-laminated members. Laminations in such members are thin enough to season readily in kiln drying without developing checks. Since checks lie in a radial plane, and the majority of laminations are essentially flat grain, checks are so positioned in horizontally laminated members that they will not materially affect shear strength. When members are designed with laminations vertical (with wide face parallel to the direction of load application), and when checks may affect the shear strength, the effect of checks may be evaluated in the same manner as for checks in solid-sawn members.

Seasoning checks in bending members affect only the horizontal shear strength. They are usually not of structural importance unless the checks are significant in depth and occur in the midheight of the member near the support, and then only if shear governs the design of the members. The reduction in shear strength is nearly directly proportional to the ratio of depth of check to width of beam. Checks in columns are not of structural importance unless the check develops into a split, thereby increasing the slenderness ratio of the columns.

Minor checking may be disregarded since there is an ample factor of safety in design values. The final decision as to whether shrinkage checks are detrimental to the strength requirements of any particular design or structural member should be made by a competent engineer experienced in timber construction.

11.1.3 Standard Sizes of Lumber and Timber

Details regarding dressed sizes of various species of wood are given in the grading rules of agencies that formulate and maintain such rules. Dressed sizes in Table 11.2 are from the American Softwood Lumber Standard, "Voluntary Product Standard PS20-70." 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.

11.1.4 Standard Sizes of Glued-Laminated Timber

Standard finished sizes of structural glued-laminated timber should be used to the extent that conditions permit. These standard finished sizes are based on lumber sizes given in "Voluntary Product Standard PS20-70." Other finished sizes may be used to meet the size requirements of a design or other special requirements.

Nominal 2-in-thick lumber, surfaced to $1\frac{3}{8}$ or $1\frac{1}{2}$ in before gluing, is used to laminate straight members and curved members with radii of curvature within the bending-radius limitations for the species. Nominal 1-in-thick lumber, surfaced to $\frac{5}{8}$ or $\frac{3}{4}$ in before gluing, may be used for laminating curved members when the bending radius is too short to permit use of nominal 2-in-thick laminations if the bending-radius limitations for the species are observed. Other lamination thicknesses may be used to meet special curving requirements.

11.1.5 Section Properties of Wood Members

Sectional properties of solid-sawn lumber and timber and glue-laminated timber members are shown on the web page for the American Institute of Timber Construction (AITC) and listed in AITC's "Timber Construction Manual," 4th ed., published by John Wiley & Sons (www.wiley.com).

11.2 Structural Grading of Wood

Strength properties of wood are intimately related to moisture content and specific gravity. Therefore, data on strength properties unaccompanied by corresponding data on these physical properties are of little value.

The strength of wood is actually affected by many other factors, such as rate of loading, duration of load, temperature, direction of grain, and position of growth rings. Strength is also influenced by such inherent growth characteristics as knots, cross grain, shakes, and checks.

Analysis and integration of available data have yielded a comprehensive set of simple principles for grading structural lumber.

The same characteristics, such as knots and cross grain, that reduce the strength of solid timber also affect the strength of laminated members. However, additional factors peculiar to laminated wood must be considered: Effect on strength of bending members is less from knots located at the neutral plane of the beam, a region of low stress. Strength of a bending member with low-grade laminations can be improved by substituting a few high-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 this effect may be established for members containing various numbers of laminations.

Design values taking these factors into account are higher than for solid timbers of comparable grade. But cross-grain limitations must be more restrictive than for solid timbers, to justify these higher design values.

Table 11.2 Nominal and Minimum Dressed Sizes of Boards, Dimension, and Timbers

	Thickness, in			Face Width, in		
		Minimum Dressed			Minimum Dressed	
Item	Nominal	Dry*	Green†	Nominal	Dry	Green†
Boards	1	3/4	25/32	2	1 1/2	1 9/16
	1 1/4	1	1 1/32	3	2 1/2	2 9/16
	1 1/2	1 1/4	1 9/16	4	3 1/2	3 9/16
				5	4 1/2	4 5/8
				6	5 1/2	5 5/8
				7	6 1/2	6 5/8
				8	7 1/4	7 1/2
				9	8 1/4	8 1/2
				10	9 1/4	9 1/2
				11	10 1/4	10 1/2
				12	11 1/4	11 1/2
				14	13 1/4	13 1/2
				16	15 1/4	15 1/2
Dimension	2	1 1/2	1 9/16	2	1 1/2	1 9/16
	2 1/2	2	2 1/16	3	2 1/2	2 9/16
	3	2 1/2	2 9/16	4	3 1/2	3 9/16
	3 1/2	3	3 1/16	5	4 1/2	4 5/8
				6	5 1/2	5 5/8
				8	7 1/4	7 1/2
				10	9 1/4	9 1/2
				12	11 1/4	11 1/2
				14	13 1/4	13 1/2
				16	15 1/4	15 1/2
	4	3 1/2	3 9/16	2	1 1/2	1 9/16
	4 1/2	4	4 1/16	3	2 1/2	2 9/16
				4	3 1/2	3 9/16
				5	4 1/2	4 5/8
				6	5 1/2	5 5/8
				8	7 1/4	7 1/2
				10	9 1/4	9 1/2
				12	11 1/4	11 1/2
				14		13 1/2
				16		15 1/2
Timbers	5 and thicker		1/2 in less	5 and wider		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%.

Table 11.3 Standard Nominal and Finished Widths of Glued-Laminated Timber

Nominal width of stock, in	3	4	6	8	10	12	14	16
Net finished width, in (western softwoods)	$2\frac{1}{8}$	$3\frac{1}{8}$	$5\frac{1}{8}$	$6\frac{3}{4}$	$8\frac{3}{4}$	$10\frac{3}{4}$	$12\frac{1}{4}$	$14\frac{1}{4}$
Net finished width, in (southern pine)	$2\frac{1}{8}$	3 or $3\frac{1}{8}$	5 or $5\frac{1}{8}$	$6\frac{3}{4}$	$8\frac{1}{2}$	$10\frac{3}{4}$	$12\frac{1}{4}$	$14\frac{1}{4}$

11.3 Design Values for Lumber, Timber, and Structural Glued-Laminated Timber

Testing a species to determine average strength properties should be carried out from either of two viewpoints:

1. Tests should be made on specimens of large size containing defects. Practically all structural uses involve members of this character.
2. Tests should be made on small, clear specimens to provide fundamental data. Factors to account for the influence of various characteristics may be applied to establish the design values of structural members.

Tests made in accordance with the first viewpoint have the disadvantage that the results apply only to the particular combination of characteristics existing in the test specimens. To determine the strength corresponding to other combinations requires additional tests; thus, an endless testing program is necessary. The second viewpoint permits establishment of fundamental strength properties for each species and application of general rules to cover the specific conditions involved in a particular case.

This second viewpoint has been generally accepted. When a species has been adequately investigated under this concept, there should be no need for further tests on that species unless new conditions arise.

Basic stresses are essentially unit stresses applicable to clear and straight-grained defect-free material. These stresses, derived from the results of tests on small, clear specimens of green wood, include an adjustment for variability of material, length of loading period, and factor of safety. They are considerably less than the average for the species. They require only an adjustment for grade to become allowable unit stresses.

Allowable unit stresses are computed for a particular grade by reducing the basic stress according to the limitations on defects for that grade. The basic stress is multiplied by a strength ratio to obtain an allowable stress. This strength ratio represents that proportion of the strength of a defect-free piece that remains after taking into account the effect of strength-reducing features.

The principal factors entering into the establishment of allowable unit stress for each species include inherent strength of wood, reduction in strength due to natural growth characteristics permitted in the grade, effect of long-time loading, variability of individual species, possibility of some slight overloading, characteristics of the species, size of member and related influence of seasoning, and factor of safety. The effect of these factors is a strength value for practical-use conditions lower than the average value taken from tests on small, clear specimens.

When moisture content in a member will be low throughout its service, a second set of higher basic stresses, based on the higher strength of dry material, may be used. Technical Bulletin 479, U.S. Department of Agriculture, "Strength and Related Properties of Woods Grown in the United States," presents tests results on small, clear, and straight-grained wood species in the green state and in the 12%-moisture-content, air-dry condition.

Design values for an extensive range of sawn lumber and timber are tabulated in "National Design Specification for Wood Construction," (NDS), American Forest and Paper Association (AFPA), 1111 19th St., N. W., Suite 800, Washington, DC 20036 (www.afandapa.org).

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. Design values for most species and grades of visually graded dimension lumber are based on provisions in "Establishing Allowable Properties for Visually Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens," ASTM D1990. 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. This standard 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.

Glued-Laminated Timber ▪ Design values for glued-laminated timber, developed by the American Institute of Timber Construction (AITC) and published by American Wood Systems (AWS) in accordance with principles originally established by the U.S. Forest Products Laboratory, are included in the NDS. The principles are the basis for the "Standard Method for Establishing Stresses for Structural Glued-Laminated Timber (Glulam)," ASTM D3737. It requires determination of the strength properties of clear, straight-grained lumber in accordance with the methods of ASTM D2555 or as given in a table in D3737. The ASTM test method also specifies procedures for obtaining design values by adjustments to those properties to account for the effects of knots, slope of grain, density, size of member, curvature, number of laminations, and other factors unique to laminating.

See also Art. 11.4.

11.4 Adjustment Factors for Design Values

Design values obtained by the methods described in Art. 11.2 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

$$F_b' = F_b C_D C_M C_t C_L C_F C_V C_r C_c \tag{11.1}$$

where F_b = design value for extreme-fiber bending

C_D = load-duration factor (Art. 11.4.2)

C_M = wet-service factor (Art. 11.4.1)

C_t = temperature factor (Art. 11.4.3)

C_L = beam stability factor (Arts. 11.4.6 and 11.5)

C_F = size factor—applicable only to visually graded, sawn lumber and round timber flexural members (Art. 11.4.4)

C_V = volume factor—applicable only to glued-laminated beams (Art. 11.4.4)

C_r = repetitive-member factor—applicable only to dimension-lumber beams 2 to 4 in thick (Art. 11.4.9)

C_c = curvature factor—applicable only to curved portions of glued-laminated beams (Art. 11.4.8)

For glued-laminated beams, use either C_L or C_V, whichever is smaller, not both, in Eq. (11.1).

The adjusted design value for tension F_t' is given by

$$F_t' = F_t C_D C_M C_t C_F \tag{11.2}$$

where F_t = design value for tension.

For shear, the adjusted design value F_V' is computed from

$$F_V' = F_V C_D C_M C_t C_H \tag{11.3}$$

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. 11.4.12).

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

$$F'_{c\perp} = F_{c\perp} C_M C_t C_b \tag{11.4}$$

where $F_{c\perp}$ = design value for compression perpendicular to the grain and C_b = bearing area factor (Art. 11.4.10).

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

$$F'_c = F_c C_D C_M C_t C_F C_P \tag{11.5}$$

where F_c = design value for compression parallel to grain and C_P = column stability factor (Arts. 11.4.11 and 11.11).

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

$$F'_g = F_g C_D C_t \tag{11.6}$$

where F_g = design value for end grain in bearing parallel to the grain. See also Art. 11.14.

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

$$E' = E C_M C_T C \ldots \tag{11.7}$$

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 ⅜ in or more thick is nailed to the narrow face (Art. 11.4.11).

$C\ldots$ = other appropriate adjustment factors

11.4.1 Wet-Service Factor

As indicated in Art 11.1.1, design values should be adjusted for moisture content.

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 structural members in service will exceed 19% for an extended period of time, design values should be multiplied by the appropriate wet-service factor listed in Table 11.4.

Table 11.4 Wet-Service Factors C_M

Design Value	C_M for Sawn Lumber*	C_M for Glulam Timber†
F_b	0.85‡	0.80
F_t	1.0	0.80
F_V	0.97	0.875
$F_{c\perp}$	0.67	0.53
F_c	0.80§	0.73
E	0.90	0.833

* 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.

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%.

Glued-laminated design values apply when the MC in service is less than 16%, as in most covered structures. When MC is 16% or more, design values should be multiplied by the appropriate wet-service factor C_M in Table 11.4.

11.4.2 Load-Duration Factor

Wood can absorb overloads of considerable magnitude for short periods; thus, allowable unit stresses are adjusted accordingly. The elastic limit and ultimate strength are higher under short-time loading. Wood members under continuous loading for years will fail at loads one-half to three-fourths as great as would be required to produce failure in a static-bending test when the maximum load is reached in a few minutes.

Normal load duration contemplates fully stressing a member to the allowable unit stress by the application of the full design load for a duration of about 10 years (either continuously or cumulatively). 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 11.5.

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 or structural members pressure-treated with fire retardants or with waterborne preservatives to the heavy retention required for marine exposure.

Table 11.5 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

11.4.3 Temperature Factor

Tests show that wood increases in strength as temperature is lowered below normal. Tests conducted at about $-300\,^\circ$F indicate that the important strength properties of dry wood in bending and compression, including stiffness and shock resistance, are much higher at extremely low temperatures.

Some reduction of the design values for wood may be necessary for members subjected to elevated temperatures for repeated or prolonged periods. This adjustment is especially desirable where high temperature is associated with high moisture content.

Temperature effect on strength is immediate. Its magnitude depends on the moisture content of the wood and, when temperature is raised, the duration of exposure.

Between 0 and 70 °F, the static strength of dry wood (12% moisture content) roughly increases from its strength at 70 °F about ⅓ to ½% for each 1 °F decrease in temperature. Between 70 and 150 °F, the strength decreases at about the same rate for each 1 °F increase in temperature. The change is greater for higher wood moisture content.

After exposure to temperatures not much above normal for a short time under ordinary atmospheric conditions, the wood, when temperature is reduced to normal, may recover essentially all its original strength. Experiments indicate that air-dry wood can probably be exposed to temperatures up to nearly 150 °F for a year or more without a significant permanent loss in most strength properties. But its strength while at such temperatures will be temporarily lower than at normal temperature.

When wood is exposed to temperatures of 150 °F or more for extended periods of time, it will be permanently weakened. The nonrecoverable strength loss depends on a number of factors, including moisture content and temperature of the wood, heating medium, and time of exposure. To some extent, the loss depends on the species and size of the piece.

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 11.6.

Glued-laminated members are normally cured at temperatures of less than 150 °F. Therefore, no reduction in allowable unit stresses due to temperature effect is necessary for curing.

Adhesives used under standard specifications for structural glued-laminated members, for example, casein, resorcinol-resin, phenol-resin, and melamine-resin adhesives, are not affected substantially by temperatures up to those that char wood. Use of adhesives that deteriorate at high temperatures is not permitted by standard specifications for structural glued-laminated timber. Low temperatures appear to have no significant effect on the strength of glued joints.

Table 11.6 Temperature Factors C_t

Design Values and In-Service Moisture Conditions	$T \leq 100°F$	$100°F < T \leq 125°F$	$125°F < T \leq 150°F$
F_t and E, wet or dry	1.0	0.09	0.9
F_b, F_V, F_c, and $F_{c\perp}$			
Dry	1.0	0.8	0.7
Wet	1.0	0.7	0.5

Modifications for Pressure-Applied Treatments ▪ The design values given for wood also apply to wood treated with a preservative when this treatment is in accordance with American Wood Preservers Association (AWPA) standard specifications, which limit pressure and temperature. Investigations have indicated that, in general, any weakening of timber as a result of preservative treatment is caused almost entirely by subjecting the wood to temperatures and pressures above the AWPA limits.

The effects on strength of all treatments, preservative and fire-retardant, should be investigated, to ensure that adjustments in design values are made when required ("Manual of Recommended Practice," American Wood Preservers Association).

11.4.4 Size and Volume Factors

For visually graded dimension lumber, design values F_b, F_t, and F_c for all species and species combinations, except southern pine, should be multiplied by the appropriate size factor C_F given in Table 11.7 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. This equation based on in-grade test data, accounts 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

Table 11.7 Size Factors C_F

	F_b				
		Thickness, in			
Grades	Width, in	2 and 3	4	F_t	F_c
Select Structural	2, 3, and 4	1.5	1.5	1.5	1.15
No. 1 and better	5	1.4	1.4	1.4	1.1
No. 1, No. 2,	6	1.3	1.3	1.3	1.1
No. 3	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

exceeds 12 in, the design value for bending should be adjusted by the size factor

$$C_F = \left(\frac{12}{d}\right)^{1/9} \tag{11.8}$$

Design values for bending F_b for glued-laminated beams should be adjusted for the effects of volume by multiplying by

$$C_V = K_L\left[\left(\frac{5.125}{b}\right)^{1/x}\left(\frac{12}{d}\right)^{1/x}\left(\frac{21}{L}\right)^{1/x}\right] \tag{11.9}$$

where L = length of beam between inflection points, ft
d = depth, in, of beam
b = width, in, of beam
x = 20 for southern pine
= 10 for other species
K_L = loading condition coefficient (Table 11.8)

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

Table 11.8 Loading-Condition Coefficient K_L for Glued-Laminated 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 or Cantilevers	
All loading conditions	1.00

11.4.5 Beam Stability Factor

Design values F_b for bending should be adjusted by multiplying by the beam stability factor C_L specified in Art. 11.5. For glued-laminated beams, the smaller value of C_L and the volume factor C_V should be used, not both. See also Art. 11.4.4.

11.4.6 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.

11.4.7 Curvature Factor

The radial stress induced by a bending moment in a member of constant cross section may be computed from

$$f_r = \frac{3M}{2Rbd} \tag{11.10}$$

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 design value is limited to that for compression perpendicular to grain for all species.

For the curved portion of members, the design value for wood in bending should be modified by multiplication by the following curvature factor:

$$C_c = 1 - 2000\left(\frac{t}{R}\right)^2 \tag{11.11}$$

where t = thickness of lamination, in

R = radius of curvature of lamination, in

t/R should not exceed $\frac{1}{100}$ for hardwoods and southern pine, or $\frac{1}{125}$ for softwoods other than southern pine. The curvature factor should not be applied to stress in the straight portion of an assembly, regardless of curvature elsewhere.

The recommended minimum radii of curvature for curved, structural glued-laminated members of Douglas fir are 9 ft 4 in for $\frac{3}{4}$-in laminations, and 27 ft 6 in for $1\frac{1}{2}$-in laminations. Other radii of curvature may be used with these thicknesses, and other radius-thickness combinations may be used.

Certain species can be bent to sharper radii, but the designer should determine the availability of such sharply curved members before specifying them.

11.4.8 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 $C_r = 1.15$.

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.

11.4.9 Bearing Area Factor

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

$$C_b = \frac{L_b + 0.375}{L_b} \tag{11.12}$$

where L_b = bearing length, in, measured parallel to grain. Equation (11.12) yields the values of C_b for elements with small areas, such as plates and washers, listed in Table 11.9. For round bearing areas, such as washers, L_b should be taken as the diameter.

11.4.10 Column Stability and Buckling Stiffness Factors

Design values for compression parallel to the grain F_c should be multiplied by the column stability factor C_P given by Eq. (11.13).

$$C_P = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{(F_{cE}/F_c^*)}{c}} \tag{11.13}$$

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

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

Table 11.9 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

E' = modulus of elasticity multiplied by adjustment factors

K_{cE} = 0.3 for visually graded lumber and machine-evaluated lumber

= 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 glued-laminated timber

For a compression member braced in all directions throughout its length to prevent lateral displacement, $C_P = 1.0$. See also Art. 11.11.

The buckling stiffness of a truss compression chord of sawn lumber subjected to combined flexure and axial compression under dry service conditions may be increased if the chord is 2 × 4 in or smaller and has the narrow face braced by nailing to plywood sheathing at least ⅜ 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

$$C_T = 1 + \frac{K_M L_e}{K_T E} \tag{11.14}$$

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 and machine-evaluated lumber

= 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. (11.14) with $L_e = 96$ in. For additional information on wood trusses with metal-plate connections, see design standards of the Truss Plate Institute, Madison, Wisconsin.

11.4.11 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. 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 Specification for Wood Construction," American Forest and Paper Association, lists values of C_H for lumber and timber of various species.

11.5 Lateral Support of Wood Framing

To prevent beams and compression members from buckling, they may have to be braced laterally. Need for such bracing and required spacing depend on the unsupported length and cross-sectional dimensions of members.

When buckling occurs, a member deflects in the direction of its least dimension b, unless prevented by bracing. (In a beam, b usually is taken as the width.) But if bracing precludes buckling in that direction, deflection can occur in the direction of the perpendicular dimension d. Thus, it is logical that unsupported length L, b, and d play important roles in rules for lateral support, or in formulas for reducing allowable stresses for buckling.

For flexural members, design for lateral stability is based on a function of Ld/b^2. For solid-sawn beams of rectangular cross section, maximum depth-width ratios should satisfy the approximate rules, based on nominal dimensions, summarized

Table 11.10 Approximate Lateral-Support Rules for Lumber Beams*

Depth-Width Ratio (Nominal Dimensions)	Rule
2 or less	No lateral support required
3	Hold ends in position
4	Hold ends in position and member in line, e.g., with purlins or sag rods
5	Hold ends in position and compression edge in line, e.g., with direct connection of sheathing, decking, or joists
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 6 times the depth
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 dead load is sufficient to induce tension on the underside of the rafters, the ratio for the beam may be 6:1.

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

in Table 11.10. When the beams are adequately braced laterally, the depth of the member below the brace may be taken as the width.

No lateral support is required when the depth does not exceed the width. In that case also, the design value does not have to be adjusted for lateral instability. Similarly, if continuous support prevents lateral movement of the compression flange, lateral buckling cannot occur and the design value need not be reduced.

When the depth of a flexural member exceeds the width, bracing must be provided at supports. This bracing must be so placed as to prevent rotation of the beam in a plane perpendicular to its longitudinal axis. Unless the compression flange is braced at sufficiently close intervals between the supports, the design value should be adjusted for lateral buckling.

The slenderness ratio R_B for beams is defined by

$$R_B = \sqrt{\frac{L_e d}{b^2}} \qquad (11.15)$$

The slenderness ratio should not exceed 50.

The effective length L_e for Eq. (11.15) is given in terms of unsupported length of beam in Table 11.11. 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.

Acceptable methods of providing adequate bracing at supports include anchoring the bottom of a beam to a pilaster and the top of the beam to a parapet; for a wall-bearing roof beam, fastening the roof diaphragm to the supporting wall or installing a girt between beams at the top of the wall; for beams on wood columns, providing rod bracing.

For continuous lateral support of a compression flange, composite action is essential between deck elements, so that sheathing or deck acts as a diaphragm. One example is a plywood deck with edge nailing. With plank decking, nails attaching the plank to the beams must form couples, to resist rotation. In addition, the planks must be nailed to each other, for diaphragm action. Adequate lateral support is not provided when only one nail is used per plank and no nails are used between planks.

The beam stability factor C_L may be calculated from

$$C_L = \frac{1 + (F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1 + (F_{bE}/F_b^*)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}} \qquad (11.16)$$

Table 11.11 Effective Length L_e for Lateral Stability of Beams*

Loading	For Depth Greater than Width†	For Loads from Secondary Framing‡
	Simple Beam§	
Uniformly distributed load	$1.63L_u + 3d$	
Load concentrated at midspan	$1.37L_u + 3d$	$1.11L_u$
Equal end moments	$1.84L_u$	
Equal concentrated loads at third points		$1.68L_u$
Equal concentrated loads at quarter points		$1.54L_u$
Equal concentrated loads at fifth points		$1.68L_u$
	Cantilever§	
Uniformly distributed load	$0.90L_u + 3d$	
Concentrated load on the end	$1.44L_u + 3d$	

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

† 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.

‡ 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 $1.63L_u + 3d$ when $L_u/d > 14.3$ and $1.84L_u$ when $L_u/d > 14.3$.

where F_b^* = design value for bending multiplied by all applicable adjustment factors except C_{fu}, C_V, and C_L (Art. 11.4)

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

K_{bE} = 0.438 for visually graded lumber and machine-evaluated lumber

= 0.609 for products with a coefficient of variation of 0.11 or less

E' = design modulus of elasticity multiplied by applicable adjustment factors (Art. 11.4)

(American Institute of Timber Construction (www.aitc-glulam.org), "Timber Construction Manual," 4th ed., John Wiley & Sons, Inc., New York (www.wiley.com); "National Design Specification," American Forest and Paper Association (www.afandpa.org); "Western Woods Use Book," Western Wood Products Association, 522 S.W. Fifth Ave., Portland, OR 97204 (www.wwpa.org).)

11.6 Manufacture of Glued-Laminated Lumber

Structural glued-laminated lumber is made by bonding together layers of lumber with adhesive so that the grain direction of all laminations is essentially parallel. Narrow boards may be edge-glued; short boards, end-glued; and the resultant wide and long laminations then face-glued into large, shop-grown timbers.

Recommended practice calls for lumber of nominal 1- and 2-in thicknesses for laminating. The thinner laminations are generally used in curved members.

Depth of constant-depth members normally is a multiple of the thickness of the lamination stock used. Depths of variable-depth members, due to tapering or special assembly techniques, may not be exact multiples of these lamination thicknesses.

Industry-standard finished widths correspond to the nominal widths in Table 11.3 after allowance for drying and surfacing of nominal lumber widths. Standard widths are most economical since they represent the maximum width of board normally obtained from the lumber stock used in laminating.

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. The objective when creating long laminations required from lumber of shorter lengths is to avoid butt joints at the lumber ends. Wood, being of a hollow tube structure, does not bond well end to end.

Edge and face gluings are the simplest to make, end gluings the most difficult. Ends are also the most difficult surfaces to machine. Scarfs or finger joints generally are used to avoid end gluing.

A plane sloping scarf (Fig. 11.2), 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. A relatively flat slope on the plane scarf or on the individual slopes of the finger joint provide gluing surfaces that can give high shear resistance to a tension parallel to grain force along the lamination. Finger joints (Fig. 11.3) are less wasteful of lumber. 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. In testing the quality of glued end joints, the objective is failure to occur in the wood as opposed to adhesive failure.

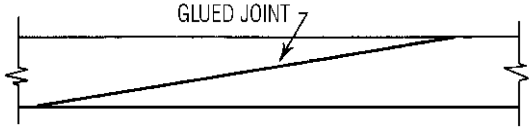

Fig. 11.2 Plane sloping scarf.

The usefulness of structural glued-laminated timbers is determined by the lumber used and glue joint produced. Certain combinations of adhesive, treatment, and wood species do not produce the same quality of glue bond as other combinations, although the same gluing procedures are used.

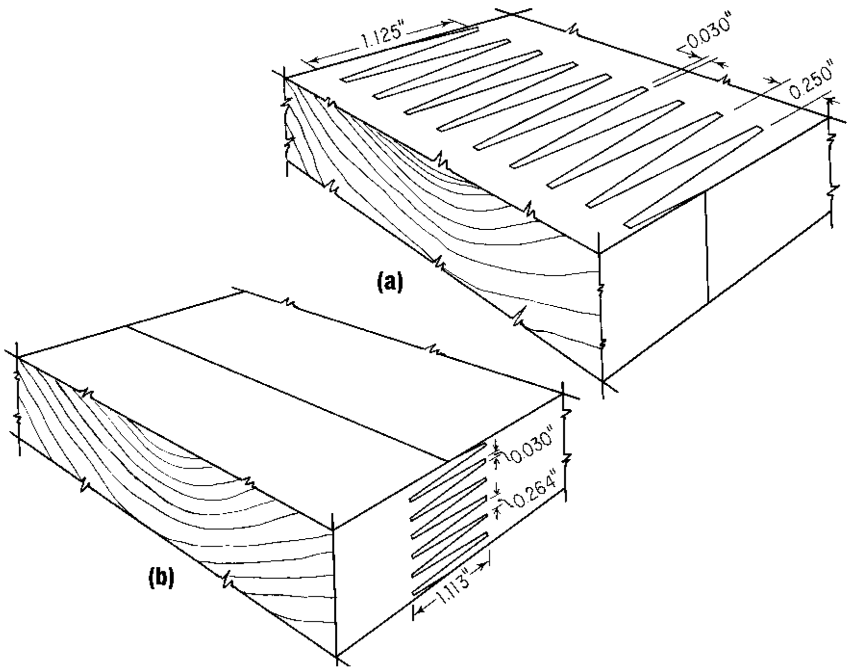

Fig. 11.3 Finger joint: (*a*) Fingers formed by cuts perpendicular to the wide face of the board; (*b*) fingers formed by cuts perpendicular to the edges.

Thus, a combination must be supported by adequate experience with a laminator's gluing procedure (see also Art. 11.25).

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

Glued joints are cured with heat by several methods. R. F. (high-frequency) 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, is used for attaching thin facing pieces. The metal may 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 temperature 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 temperatures is avoided.

11.7 Fabrication of Structural Timber

Fabrication consists of boring, cutting, sawing, trimming, dapping, routing, planing, and otherwise shaping, framing, and furnishing wood units, sawn or laminated, including plywood, 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 work.

Jigs, patterns, templates, stops, or other suitable means should be used for all complicated and multiple assemblies to insure accuracy, uniformity, and control of all dimensions. All tolerances in cutting, drilling, and framing must comply with good practice in the industry 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, and so on specified in the "National Design Specification for Wood," American Forest and 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 $1/16$ in/ft of depth and width. Square or sloped ends to be loaded in compression should be cut to provide contact over substantially the complete surface.

Shrinkage or Swelling Effects on Shape of Curved Members ▪ Wood shrinks or swells across the grain but has practically no dimensional change along the grain. Radial swelling causes a decrease in the angle between the ends of a curved member; radial shrinkage causes an increase in this angle.

Such effects may be of great importance in three-hinged arches that become horizontal, or nearly so, at the crest of a 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 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.

11.8 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.

On receipt at the site, 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. And the erector must see that all supports and anchors are complete, accessible, and free from obstructions.

Jobsite 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, and especially glued-laminated members, stored at the site should be set above the ground on appropriate blocking. The members should be separated with strips so that air may circulate around all sides of each member. 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 jobsite debris. (Do not use clear polyethylene films 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.

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. Although 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. Used specifically for protection in transit, the paper should remain in place until the roof covering is in place. It may be necessary, however, to remove the paper from isolated areas to make connections from one member to another. If temporarily removed, the paper should be replaced and should remain in position until all the wrapping may be removed.

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 or crane. Do not dump, drag, or drop members.

During unloading with lifting equipment, use fabric or plastic belts, or other slings that will not mar the wood. If chains or cables are used, provide protective blocking or padding.

Equipment ▪ Adequate equipment of proper load-handling capacity, with control for moving and placing 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, cables, or 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 normal 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 jobsite 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 permanently braced in the building system.

Assembly and Subassembly ▪ Whether these are done in a shop or on the ground or in the air in the field depends 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 for the materials. Any additional drilling or dapping, as well as the installation of all field connections, must be done in a workmanlike manner.

Trusses are usually 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.

Prior to 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. 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 restraint or support, or both, to insure 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. It should be attached so that children and other casual visitors cannot remove it or prevent it from serving as intended. Protective corners and other protective devices should be installed to prevent members from being damaged by the bracing.

In timber-truss construction, temporary bracing can be used to plumb trusses during erection and hold them in place until they receive the rafters and roof sheathing. The major portion of temporary bracing for trusses is left in place because it is designed to brace the completed 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.

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.

The magnitude of the restraining force that should be provided by a cable guy or brace cannot be precisely determined, but 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.

11.9 Design Recommendations

The following recommendations aim at achieving economical designs with wood framing:

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

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, and thus avoid design, erection, 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.

Consider using roof profiles that favorably influence the type and amount of load on the structure.

Specify design values rather than the lumber grade or combination of grades to be used.

Select an adhesive suitable for the service conditions, but do not overspecify. For example, waterproof resin adhesives need not be used where less expensive water-resistant adhesives will do the job.

Use lumber treated with preservatives where service conditions dictate. Such treatment need not be used where decay 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 already a low fire risk.

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 premium appearance grade for all members if it is not required.

Table 11.12 is a guide to economical span ranges for roof and floor framing in buildings.

Designing for Fire Safety ▪ Maximum protection of the occupants of a building and the property itself can be achieved in timber design by taking advantage of the fire-endurance properties of wood in large cross sections and by close attention to details that make a building fire-safe. Building materials alone, building features alone, or detection and fire-extinguishing equipment alone cannot provide maximum safety from fire in buildings. A proper combination of these three factors will provide the necessary degree of protection for the occupants and the property.

Table 11.12 Economical Span Range for Framing Members

Framing Member	Economical Span Range, ft	Usual Spacing, ft
Roof beams (generally used where a flat or low-pitched roof is desired):		
Simple span:		
Constant depth		
Solid-sawn	0–40	4–20
Glued-laminated	20–100	8–24
Tapered	25–100	8–24
Double tapered (pitched beams)	25–100	8–24
Curved beams	25–100	8–24
Simple beam with overhangs (usually more economical than simple span when span is over 40 ft):		
Solid-sawn	24	4–20
Glued-laminated	10–90	8–24
Continuous span:		
Solid-sawn	10–50	4–20
Glued-laminated	10–50	8–24
Arches (three-hinged for relatively high-rise applications and two-hinged for relatively low-rise applications):		
Three-hinged:		
Gothic	40–90	8–24
Tudor	30–120	8–24
A-frame	20–160	8–24
Three-centered	40–250	8–24
Parabolic	40–250	8–24
Radial	40–250	8–24
Two-hinged:		
Radial	50–200	8–24
Parabolic	50–200	8–24
Trusses (provide openings for passage of wires, piping, etc.)		
Flat or parallel chord	50–150	12–20
Triangular or pitched	50–90	12–20
Bowstring	50–200	14–24
Tied arches (where no ceiling is desired and where a long, clear span is desired with low rise):		
Tied segment	50–100	8–20
Buttressed segment	50–200	14–24
Domes	50–350	8–24
Simple-span floor beams:		
Solid-sawn	6–20	4–12
Glued-laminated	6–40	4–16
Continuous floor beams	25–40	4–16
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	
Plywood sheathing	1–4	
Sheathing on roof joists	1.33–2	
Plank floor decking (floor and ceiling in one):		
Edge to edge	4–16	
Wide face to wide face	4–16	

The following should be investigated:

Degree of protection needed, as dictated by occupancy or operations taking place

Number, size, type (such as direct to the outside), and accessibility of exits (particularly stairways), and their distance from each other

Installation of automatic alarm and sprinkler systems

Separation of areas in which hazardous processes or operations take place, such as boiler rooms and workshops

Enclosure of stairwells and use of self-closing fire doors

Fire stopping and elimination, or proper protection of concealed spaces

Interior finishes to assure surfaces that will not spread flame at hazardous rates

Roof venting equipment or provision of draft curtains where walls might interfere with production operations

When exposed to fire, wood forms a self-insulating surface layer of char, which provides its own fire protection. Even though the surface chars, the undamaged wood beneath retains 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 the charred surfaces have been refinished. 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 contents and create conditions that render the inside of the structure uninhabitable long before the structural framing becomes involved in the fire. Thus, whether the building is classified as combustible or noncombustible 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 significantly help facilitate evacuation, fire fighting, and property protection.

The most important protection factors for occupants, firefighters, and the property, as well as adjacent exposed property, are prompt detection of the fire, immediate alarm, and rapid extinguishment of the fire. Firefighters do not fear fires in buildings of heavy-timber construction as they do those in buildings of many other types of construction. They need not fear sudden collapse without warning; they usually have adequate time, because of the slow-burning characteristics of the timber, to ventilate the building and fight the fire from within the building or on top.

With size of member of particular importance to fire endurance of wood members, building codes specify minimum dimensions for structural members and classify buildings with wood framing as heavy-timber construction, ordinary construction, or wood-frame construction.

Heavy-timber construction is that type in which fire resistance is attained by placing limitations 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 fastenings, construction details, and adhesives; and by providing the required degree of fire resistance in exterior and interior walls. (See AITC 108, "Heavy Timber Construction," American Institute of Timber Construction.)

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 interior finish on exposed surfaces of rooms, corridors, and stairways is important from the standpoint of its tendency to ignite, flame, and spread fire from one location to another. The fact that wood is combustible does not mean that it will spread flame at a hazardous rate. Most codes exclude the exposed wood surfaces of heavy-timber structural members from flame-spread requirements because such wood is difficult to ignite and, even with an external source of heat, such as burning contents, is resistant to spread of flame.

Fire-retardant chemicals may be impregnated in wood with recommended retentions to lower the rate of surface flame spread and make the wood self-extinguishing if the external source of heat is removed. After proper surface preparation, the surface is paintable. Such treatments are accepted under several specifications, including federal government 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 alternate to noncombustible secondary members and decking meeting the requirements of Underwriters' Laboratories, Inc., NM 501 or NM 502, nonmetallic roof-deck assemblies in otherwise heavy-timber construction.

11.10 Wood Tension Members

The tensile stress f_t parallel to the grain should be computed from P/A_n, where P is the axial load and A_n is the net section area. This stress should not exceed the design value for tension parallel to grain f_t, adjusted as required by Eq. (11.2).

Tensile stress perpendicular to the grain should be avoided as there are no such allowable design values for this condition.

11.11 Wood Columns

Wood compression members may be a solid piece of lumber or timber (Fig. 11.4*a*), or spaced columns, connector-joined (Fig. 11.4*b* and *c*), or built-up (Fig. 11.4*d*).

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,

$$f_c = \frac{P}{A_g} \leq F'_c \tag{11.17}$$

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 given by Eq. (11.13)

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

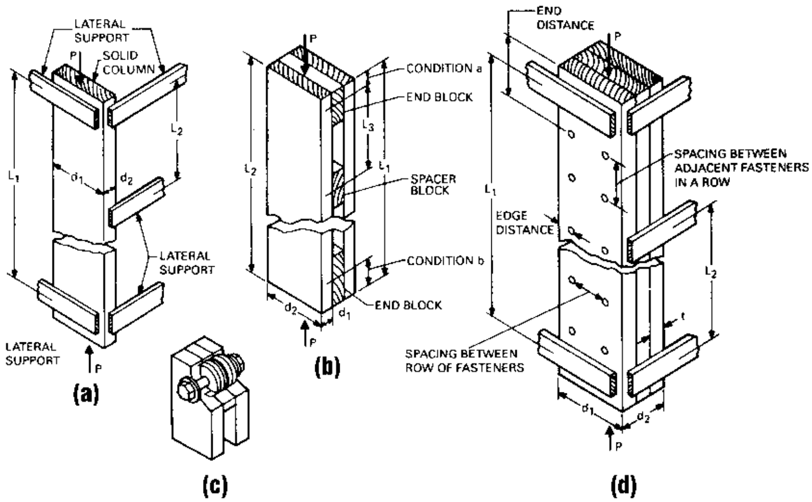

Fig. 11.4 Bracing of wood columns to control length-thickness and depth-thickness ratios: (*a*) For a solid wood column; (*b*) For a spaced column (the end distance for condition *a* should not exceed $L_1/20$ and for condition *b* should be between $L_1/20$ and $L_1/10$). (*c*) Shear plate connection in the end block of the spaced column. (*d*) Bracing for a built-up column. (*From F. S. Merritt and J. T. Ricketts, "Building Design and Construction Handbook," 5th ed., McGraw-Hill Publishing Company, New York.*)

exceed F_c, the design value for compression parallel to grain, multiplied by applicable adjustment factors, except C_P; that is,

$$f_c = \frac{P}{A_n} \leq F_c \tag{11.18}$$

where A_n = net cross-sectional area.

C_P 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 length L_e may be taken as the actual column length multiplied by the appropriate buckling-length coefficient K_e indicated in Fig. 9.5, p. 9.18. For the column in Fig. 11.4*a*, 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 critical section of columns supporting trusses frequently exists at the connection of knee brace to column. Where no knee brace is used, or the column supports a beam, the critical section for moment usually occurs at the bottom of truss or beam. Then, a rigid connection must be provided to resist moment, or adequate diagonal bracing must be provided to carry wind loads into a support.

(American Institute of Timber Construction (www.aitc.org), "Timber Construction Manual," John Wiley & Sons, Inc., New York (www.wiley.com); "National Design Specification for Wood Construction," American Forest and Paper Association, 1111 19th St., N. W., Washington, DC 20036 (www.afandpa.org).)

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. 11.4*d*). 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 and 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.

Spaced Columns ▪ These 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 split-ring or shear-plate connectors at the end blocks (Fig. 11.4*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 2½ 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.

11.12 Design of Wood Flexural Members

Standard beam formulas for bending, shear, and deflection may be used to determine beam and joist sizes. Ordinarily, deflection governs design, but for short, heavily loaded beams, shear is likely to control. Bracing for beam stability is discussed in Art. 11.5. Bearing on beams is treated in Art. 11.14.

Joists are relatively narrow beams, usually spaced 12 to 24 in c to c. They generally are topped with sheathing and braced with diaphragms or cross bridging at intervals up to 10 ft. For joist spacings of 16 to 24 in c to c, 1-in sheathing usually is required. For spacings over 24 in, 2 in or more of wood decking is necessary.

Figure 11.5 shows the types of beams commonly produced in timber. Straight and single- and double-tapered straight beams can be furnished solid-sawn or glued-laminated. The curved surfaces can be furnished only glued-laminated. Beam names describe the top and bottom surfaces of the beam: The first part describes the top surface, the word following the hyphen the bottom. Sawn surfaces on the tension side of a beam should be avoided.

Table 11.13 gives the load-carrying capacity for various cross-sectional sizes of glued-laminated, simply supported beams.

Example ▪ Design a straight, glued-laminated beam, simply supported and uniformly loaded: span, 28 ft; spacing, 9 ft c to c; live load, 30 lb/ft^2; dead load, 5 lb/ft^2 for deck and 7.5 lb/ft^2 for roofing. Allowable bending stress of combination grade is 2400 psi, with modulus of elasticity $E = 1{,}800{,}000$ psi. Deflection limitation is $L/180$, where L is the span, ft. Assume the beam is laterally supported by the deck throughout its length and held in line at the ends.

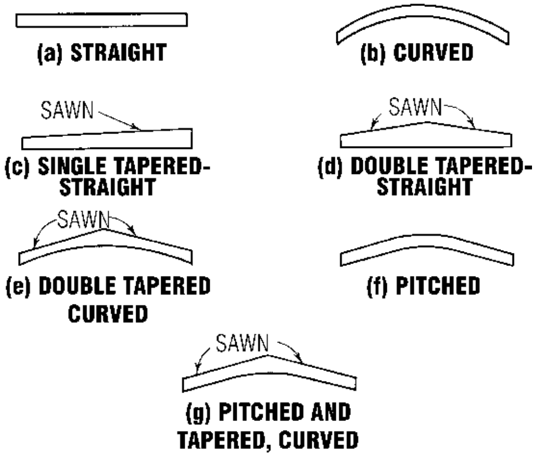

Fig. 11.5 Types of timber beams.

With a 15% increase for short-duration loading, the allowable bending stress F_b becomes 2760 psi and the allowable horizontal shear F_v, 230 psi.

Assume the beam will weigh 22.5 lb/lin ft, averaging 2.5 lb/ft^2. Then, the total uniform load comes to 45 lb/ft^2. 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, in^2, for horizontal shear is

$$A = \frac{3wL}{4F_v} = \frac{wL}{306.7} = \frac{405 \times 28}{306.7} = 37.0$$

The required section modulus, in^3, is

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

If $D = 180$, the reciprocal of the deflection limitation, then the maximum deflection equals $5 \times 1728wL^4/384EI \leq 12L/D$, where I is the moment of inertia of the beam cross section, in^4. Hence, to control deflection, the moment of inertia must be at least

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

Assume that the beam will be fabricated with $1\frac{1}{2}$-in laminations. The most economical section satisfying all three criteria is $5\frac{1}{8} \times 16\frac{1}{2}$, with $A = 84.6$, $S = 232.5$, and $I = 1918.5$. But it has a volume factor of 0.97, so the allowable bending stress must be reduced to $2760 \times 0.97 = 2677$ psi. And the required section modulus must be increased accordingly to $172.6/0.97 = 178$. Nevertheless, the selected section still is adequate.

Suspended-Span Construction ▪ Cantilever systems may comprise any of the various types and combinations of beam illustrated in Fig. 11.6. 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

Table 11.13 Load-Carrying Capacity of Simple-Span Laminated Beams*

Span, ft	Spacing, ft	Roof Beam Total-Load-Carrying Capacity						Floor Beams Total Load
		30 lb/ft²	35 lb/ft²	40 lb/ft²	45 lb/ft²	50 lb/ft²	55 lb/ft²	50 lb/ft²
8	4	3⅛ × 4½	3⅛ × 4½	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 6
	6	3⅛ × 4½	3⅛ × 4½	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 6
	8	3⅛ × 4½	3⅛ × 4½	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 7½
10	4	3⅛ × 4½	3⅛ × 4½	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 7½
	6	3⅛ × 4½	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 6	3⅛ × 7½	3⅛ × 7½
	8	3⅛ × 6	3⅛ × 6	3⅛ × 7½	3⅛ × 7½	3⅛ × 7½	3⅛ × 7½	3⅛ × 9
	10	3⅛ × 6	3⅛ × 7½	3⅛ × 7½	3⅛ × 7½	3⅛ × 7½	3⅛ × 9	3⅛ × 9
12	6	3⅛ × 6	3⅛ × 6	3⅛ × 7½	3⅛ × 7½	3⅛ × 7½	3⅛ × 7½	3⅛ × 9
	8	3⅛ × 6	3⅛ × 7½	3⅛ × 9	3⅛ × 9	3⅛ × 9	3⅛ × 9	3⅛ × 10½
	10	3⅛ × 7½	3⅛ × 7½	3⅛ × 9	3⅛ × 9	3⅛ × 9	3⅛ × 10½	3⅛ × 10½
	12	3⅛ × 7½	3⅛ × 9	3⅛ × 9	3⅛ × 9	3⅛ × 10½	3⅛ × 10½	3⅛ × 12
14	8	3⅛ × 7½	3⅛ × 9	3⅛ × 9	3⅛ × 9	3⅛ × 10½	3⅛ × 10½	3⅛ × 12
	10	3⅛ × 9	3⅛ × 9	3⅛ × 10½	3⅛ × 10½	3⅛ × 10½	3⅛ × 12	3⅛ × 12
	12	3⅛ × 9	3⅛ × 10½	3⅛ × 10½	3⅛ × 10½	3⅛ × 12	3⅛ × 12	3⅛ × 13½
	14	3⅛ × 10½	3⅛ × 10½	3⅛ × 12	3⅛ × 12	3⅛ × 12	3⅛ × 13½	3⅛ × 13½
16	8	3⅛ × 9	3⅛ × 9	3⅛ × 10½	3⅛ × 10½	3⅛ × 12	3⅛ × 12	3⅛ × 13½
	12	3⅛ × 10½	3⅛ × 12	3⅛ × 12	3⅛ × 12	3⅛ × 13½	3⅛ × 13½	3⅛ × 15
	14	3⅛ × 12	3⅛ × 12	3⅛ × 13½	3⅛ × 13½	3⅛ × 15	3⅛ × 15	3⅛ × 15
	16	3⅛ × 12	3⅛ × 13½	3⅛ × 13½	3⅛ × 15	3⅛ × 15	3⅛ × 16½	3⅛ × 15
18	8	3⅛ × 9	3⅛ × 10½	3⅛ × 12	3⅛ × 12	3⅛ × 12	3⅛ × 13½	3⅛ × 15
	12	3⅛ × 12	3⅛ × 12	3⅛ × 13½	3⅛ × 13½	3⅛ × 15	3⅛ × 16½	3⅛ × 16½
	16	3⅛ × 13½	3⅛ × 15	3⅛ × 15	3⅛ × 16½	5⅛ × 13½	5⅛ × 13½	5⅛ × 15
	18	3⅛ × 15	3⅛ × 15	3⅛ × 16½	3⅛ × 18	5⅛ × 15	5⅛ × 15	5⅛ × 15
20	8	3⅛ × 12	3⅛ × 12	3⅛ × 13½	3⅛ × 13½	3⅛ × 13½	3⅛ × 15	3⅛ × 16½
	12	3⅛ × 13½	3⅛ × 13½	3⅛ × 15	3⅛ × 16½	3⅛ × 16½	5⅛ × 13½	5⅛ × 15
	16	3⅛ × 15	3⅛ × 16½	3⅛ × 18	3⅛ × 18	5⅛ × 15	5⅛ × 16½	5⅛ × 18
	18	3⅛ × 16½	3⅛ × 16½	3⅛ × 18	5⅛ × 15	5⅛ × 16½	5⅛ × 16½	5⅛ × 18
22	8	3⅛ × 13½	3⅛ × 13½	3⅛ × 13½	3⅛ × 15	3⅛ × 15	3⅛ × 16½	5⅛ × 15
	12	3⅛ × 15	3⅛ × 15	3⅛ × 16½	3⅛ × 18	3⅛ × 18	5⅛ × 15	5⅛ × 16½
	16	3⅛ × 16½	3⅛ × 18	5⅛ × 15	5⅛ × 16½	5⅛ × 16½	5⅛ × 18	5⅛ × 19½
	18	3⅛ × 18	5⅛ × 15	5⅛ × 16½	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½
24	8	3⅛ × 13½	3⅛ × 15	3⅛ × 15	3⅛ × 16½	3⅛ × 16½	3⅛ × 18	5⅛ × 16½
	12	3⅛ × 16½	3⅛ × 16½	3⅛ × 18	5⅛ × 15	5⅛ × 16½	5⅛ × 16½	5⅛ × 18
	16	3⅛ × 18	5⅛ × 16½	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 21
	18	5⅛ × 15	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 21	5⅛ × 21
26	8	3⅛ × 15	3⅛ × 16½	3⅛ × 16½	3⅛ × 16½	3⅛ × 18	5⅛ × 16½	5⅛ × 18
	12	3⅛ × 18	3⅛ × 18	5⅛ × 16½	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½
	16	5⅛ × 16½	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 21	5⅛ × 22½
	18	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 21	5⅛ × 21	5⅛ × 22½

Table 11.13 (*Continued*)

Span, ft	Spacing, ft	Roof Beam Total-Load-Carrying Capacity 30 lb/ft^2	35 lb/ft^2	40 lb/ft^2	45 lb/ft^2	50 lb/ft^2	55 lb/ft^2	Floor Beams Total Load 50 lb/ft^2
28	8	3⅛ × 16½	3⅛ × 16½	3⅛ × 16½	3⅛ × 18	5⅛ × 16½	5⅛ × 16½	5⅛ × 19½
	12	3⅛ × 18	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 21
	16	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 19½	5⅛ × 21	5⅛ × 22½	5⅛ × 24
	18	5⅛ × 18	5⅛ × 19½	5⅛ × 19½	5⅛ × 21	5⅛ × 22½	5⅛ × 24	5⅛ × 24
30	8	3⅛ × 18	3⅛ × 18	5⅛ × 16½	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 21
	12	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 19½	5⅛ × 21	5⅛ × 22½
	16	5⅛ × 18	5⅛ × 19½	5⅛ × 21	5⅛ × 21	5⅛ × 22½	5⅛ × 24	5⅛ × 25½
	18	5⅛ × 19½	5⅛ × 21	5⅛ × 21	5⅛ × 22½	5⅛ × 24	5⅛ × 25½	5⅛ × 27
32	8	3⅛ × 18	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 21
	12	5⅛ × 18	5⅛ × 19½	5⅛ × 19½	5⅛ × 21	5⅛ × 21	5⅛ × 22½	5⅛ × 24
	16	5⅛ × 19½	5⅛ × 21	5⅛ × 22½	5⅛ × 22½	5⅛ × 24	5⅛ × 25½	5⅛ × 27
	18	5⅛ × 21	5⅛ × 21	5⅛ × 22½	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 28½
34	8	5⅛ × 16½	5⅛ × 18	5⅛ × 18	5⅛ × 19½	5⅛ × 19½	5⅛ × 21	5⅛ × 22½
	12	5⅛ × 19½	5⅛ × 19½	5⅛ × 21	5⅛ × 21	5⅛ × 22½	5⅛ × 24	5⅛ × 25½
	16	5⅛ × 21	5⅛ × 22½	5⅛ × 22½	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 28½
	18	5⅛ × 22½	5⅛ × 22½	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	5⅛ × 28½
36	12	5⅛ × 19½	5⅛ × 21	5⅛ × 22½	5⅛ × 22½	5⅛ × 24	5⅛ × 25½	6¾ × 25½
	16	5⅛ × 22½	5⅛ × 24	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	6¾ × 27
	18	5⅛ × 22½	5⅛ × 24	5⅛ × 25½	5⅛ × 28½	5⅛ × 30	6¾ × 27	6¾ × 28½
	20	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 30	6¾ × 27	6¾ × 28½	6¾ × 30
38	12	5⅛ × 21	5⅛ × 22½	5⅛ × 24	5⅛ × 24	5⅛ × 25½	5⅛ × 27	6¾ × 27
	16	5⅛ × 24	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	5⅛ × 30	6¾ × 28½
	18	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 30	6¾ × 27	6¾ × 28½	6¾ × 30
	20	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	6¾ × 27	6¾ × 28½	6¾ × 30	6¾ × 31½
40	12	5⅛ × 22½	5⅛ × 24	5⅛ × 24	5⅛ × 25½	5⅛ × 27	6¾ × 25½	6¾ × 28½
	16	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	6¾ × 27	6¾ × 28½	6¾ × 31½
	18	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	6¾ × 27	6¾ × 28½	6¾ × 30	6¾ × 31½
	20	5⅛ × 27	5⅛ × 28½	6¾ × 27	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33
42	12	5⅛ × 24	5⅛ × 24	5⅛ × 25½	5⅛ × 27	6¾ × 25½	6¾ × 25½	6¾ × 30
	16	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 33
	18	5⅛ × 27	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33
	20	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 34½
44	12	5⅛ × 24	5⅛ × 25½	5⅛ × 27	5⅛ × 27	6¾ × 25½	6¾ × 27	6¾ × 31½
	16	5⅛ × 27	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33
	18	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 34½
	20	5⅛ × 30	6¾ × 27	6¾ × 30	6¾ × 30	6¾ × 33	6¾ × 34½	6¾ × 36
46	12	5⅛ × 25½	5⅛ × 27	5⅛ × 28½	6¾ × 25½	6¾ × 27	6¾ × 28½	6¾ × 31½
	16	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 28½	6¾ × 31½	6¾ × 33	6¾ × 36
	18	5⅛ × 28½	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 36

(*Continued*)

Table 11.13 (*Continued*)

Span, ft	Spacing, ft	Roof Beam Total-Load-Carrying Capacity						Floor Beams Total Load
		30 lb/ft²	35 lb/ft²	40 lb/ft²	45 lb/ft²	50 lb/ft²	55 lb/ft²	50 lb/ft²
	20	5⅛ × 30	6¾ × 28½	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 36	8¾ × 34½
48	12	5⅛ × 27	5⅛ × 28½	5⅛ × 30	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 33
	16	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 30	6¾ × 31½	6¾ × 34½	6¾ × 37½
	18	5⅛ × 30	6¾ × 30	6¾ × 30	6¾ × 33	6¾ × 34½	6¾ × 36	8¾ × 34½
	20	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 34½	6¾ × 36	6¾ × 37½	8¾ × 36
50	12	5⅛ × 28½	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 34½
	16	5⅛ × 30	6¾ × 30	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 36	8¾ × 34½
	18	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 34½	6¾ × 36	8¾ × 33	8¾ × 36
	20	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 36	6¾ × 37½	8¾ × 34½	8¾ × 37½
52	12	5⅛ × 28½	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 31½	6¾ × 36
	16	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 37½	8¾ × 36
	18	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 37½	6¾ × 39	8¾ × 37½
	20	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 37½	6¾ × 39	8¾ × 36	8¾ × 39
54	12	5⅛ × 30	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 33	6¾ × 37½
	16	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 36	6¾ × 37½	8¾ × 37½
	18	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 36	6¾ × 39	8¾ × 36	8¾ × 39
	20	6¾ × 33	6¾ × 34½	6¾ × 36	6¾ × 39	8¾ × 36	8¾ × 37½	8¾ × 40½
56	12	6¾ × 28½	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 33	6¾ × 34½	8¾ × 36
	16	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 36	6¾ × 37½	8¾ × 34½	8¾ × 39
	18	6¾ × 33	6¾ × 34½	6¾ × 36	6¾ × 37½	8¾ × 34½	8¾ × 37½	8¾ × 40½
	20	6¾ × 33	6¾ × 36	6¾ × 37½	8¾ × 34½	8¾ × 37½	8¾ × 39	8¾ × 42
58	12	6¾ × 30	6¾ × 31½	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 36	8¾ × 37½
	16	6¾ × 31½	6¾ × 34½	6¾ × 36	6¾ × 37½	6¾ × 39	8¾ × 36	8¾ × 40½
	18	6¾ × 33	6¾ × 34½	6¾ × 37½	6¾ × 39	8¾ × 36	8¾ × 39	8¾ × 42
	20	6¾ × 34½	6¾ × 36	6¾ × 39	8¾ × 36	8¾ × 39	8¾ × 40½	8¾ × 43½
60	12	6¾ × 30	6¾ × 31½	6¾ × 33	6¾ × 34½	6¾ × 36	6¾ × 37½	8¾ × 39
	16	6¾ × 33	6¾ × 34½	6¾ × 36	6¾ × 39	8¾ × 36	8¾ × 37½	8¾ × 42
	18	6¾ × 34½	6¾ × 36	6¾ × 39	8¾ × 36	8¾ × 37½	8¾ × 39	8¾ × 43½
	20	6¾ × 36	6¾ × 37½	8¾ × 36	8¾ × 37½	8¾ × 40½	8¾ × 42	8¾ × 45

* This table applies to straight, simply supported, laminated timber beams. Other beam support systems may be employed to meet varying design conditions.

1. Roofs should have a minimum slope of ¼ in/ft to eliminate water ponding.

2. Beam weight must be subtracted from total load-carrying capacity. Floor beams are designed for uniform loads of 40 lb/ft² live load and 10 lb/ft² dead load.

3. Allowable stresses: Bending stress, $F_b = 2400$ psi (reduced by the volume factor for southern pine). Shear stress $F_v = 165$ psi. Modulus of elasticity $E = 1{,}800{,}000$ psi. For roof beams, F_b and F_v were increased 15% for short duration of loading.

4. Deflection limits: Roof beams—1/180 span for total load. Floor beams—1/360 span for 40 lb/ft² live load only. For preliminary design purposes only. For more complete design information, see the AITC "Timber Construction Manual."

5. Maximum shear stress increased to 270 psi for southern pine and to 270 psi for western species. Shear will not govern for single span beams.

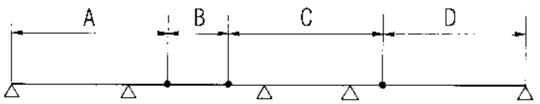

Fig. 11.6 Cantilevered-beam systems. *A* is a single cantilever; *B* is a suspended beam; *C* has a double cantilever; *D* is a beam with one end suspended.

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 should be given to deflection and camber in cantilevered multiple spans. When possible, roofs should be sloped the equivalent of ¼ in/ft of horizontal distance between the level of drains and the high point of the roof to eliminate water pockets, or provision 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.

(American Institute of Timber Construction, "Timber Construction Manual," John Wiley & Sons, Inc., New York; "National Design Specification for Wood Construction," American Forest and Paper Association, 1111 19th St., N. W., Washington, DC 20036.)

11.13 Deflection and Camber of Wood Beams

The design of many structural systems, particularly those with long spans, is governed by deflection. Strength calculations based on allowable stresses alone may result in excessive deflection. Limitations on deflection increase member stiffness.

Table 11.14 gives recommended deflection limits, as a fraction of the beam span, for wood beams. The limitation applies to live load or total load, whichever governs.

Glued-laminated beams are cambered by fabricating them with a curvature opposite in direction to that corresponding to deflections under load. Camber does not, however, increase stiffness. Table 11.15 lists recommended minimum cambers for glued-laminated timber beams.

Table 11.14 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 institutional:		
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 $l/400$	

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

† 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.

Minimum Roof Slopes ▪ Flat roofs have collapsed during rainstorms, although they were adequately designed on the basis of allowable stresses and definite deflection limitations. The reason for these collapses was the same, regardless

Table 11.15 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 + ½ applied load-deflection

* "Camber and Deflection," AITC 102, app. 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 (see under "Minimum Roof Slopes").

‡ 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.

of the structural framing used—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 ¼ in/ft between a drain and the high point of a roof, in addition to minimum recommended camber (Table 11.15), to avoid ponding. 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/ft^2 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

$$C_p = \frac{1}{1 - W'L^3/\pi^4 EI} \tag{11.19}$$

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, in^4

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

11.14 Bearing on Wood Members

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

$$f_{c\perp} = \frac{P}{A} \tag{11.20}$$

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. 11.4). (The duration-of-load factor does not apply to $F_{c\perp}$ for either solid-sawn lumber or glued laminated timber.)

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 glued laminated 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 glued laminated 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 and Paper Association), recommends that $F_{c\perp}$, psi, be reduced to $0.73F_{c\perp} + 5.60$. For glued-laminated beams, $F_{c\perp}$ may be taken as $0.73F_{c\perp}$.

Bearing stress parallel to grain f_g on a wood member 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. 11.4). 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, and it should be inserted with a snug fit between abutting ends.

Bearing perpendicular to grain 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. 11.4.10). 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.

Bearing at an angle to grain is assigned a design value that is a function of the design value F_g for bearing parallel to grain and the design value for bearing perpendicular to grain $F_{c\perp}$, which differ considerably. When load is applied at an angle θ with respect to the grain, where $0 \leq \theta \leq 90°$ (Fig. 11.7), the design value for bearing lies between F_g and $F_{c\perp}$. The "National Design Specification for Wood Construction," (American Forest and Paper Association) recommends that the design

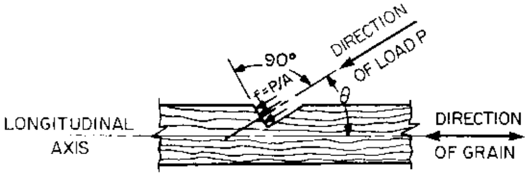

Fig. 11.7 Load applied to a member in bearing at an angle to the grain.

value for such loading be calculated from the Hankinson formula:

$$F'_n = \frac{F'_g F'_{c\perp}}{F'_g \sin^2\theta + F'_{c\perp}\cos^2\theta} \tag{11.21}$$

where F'_n = 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

11.15 Combined Stresses in Wood Members

Design values given in the "National Design Specification for Wood Construction" apply directly to bending, horizontal shear, tension parallel to grain, and compression parallel or perpendicular to grain. 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.

11.15.1 Bending and Axial Tension

Members subjected to combined bending and axial tension should be proportioned to satisfy the interaction equations, Eqs. (11.22) and (11.23).

$$\frac{f_t}{F'_t} + \frac{f_b}{F^*_b} \le 1 \tag{11.22}$$

$$\frac{(f_b - f_t)}{F^{**}_b} \le 1 \tag{11.23}$$

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. 11.4.

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.

11.15.2 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. 11.24.

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b1}}{[1-(f_c/F_{cE1})]F'_{b1}} + \frac{f_{b2}}{[1-(f_c/F_{cE2})-(f_{b1}/F_{bE})^2]F'_{b2}} \le 1 \tag{11.24}$$

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 column 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 column stability factor

For either uniaxial or biaxial bending, f_c should not exceed

$$F_{cE1} = \frac{K_{cE}E'}{(L_{e1}/d_1)^2} \tag{11.25}$$

where E' = modulus of elasticity multiplied by adjustment factors. Also, for biaxial bending, f_c should not exceed

$$F_{cE2} = \frac{K_{cE}E'}{(L_{e2}/d_2)^2} \tag{11.26}$$

and f_{b1} should not be more than

$$F_{bE} = \frac{K_{bE}E'}{R_B^2} \tag{11.27}$$

where d_1 = width of the wide face and d_2 = width of the narrow face. Slenderness ratio R_B for beams is given by Eq. (11.15). K_{bE} is defined for Eq. (11.16). 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. 11.11.

As for the case of combined bending and axial tension, F'_c, F'_{b1}, and F'_{b2} should be adjusted for duration of load by applying C_D. See Art. 11.4.

11.16 Characteristics of Mechanical Fastenings

Various kinds of mechanical fastenings are used in wood construction. The most common are nails, spikes, screws, lags, bolts, and timber connectors, such as shear plates and split rings (Art. 11.19). Joint-design data have been established by experience and tests because determination of stress distribution in wood and metal fasteners is complicated.

Design values and methods of design for bolts, connectors, and other fasteners used in one-piece sawn members also are applicable to laminated members.

Problems can arise, however, if a deep-arch base section is bolted to the shoe attached to the foundation by widely separated bolts. A decrease in wood moisture content and shrinkage will set up considerable tensile stress perpendicular to grain, and splitting may occur. If the moisture content at erection is the same as that to be reached in service, or if the bolt holes in the shoe are slotted to permit bolt movement, the tendency to split will be reduced.

Fasteners subject to corrosion or chemical attack should be protected by painting, galvanizing, or plating. In highly corrosive atmospheres, such as in chemical plants, metal fasteners and connections should be galvanized or made of stainless steel. Consideration may be given to covering connections, with hot tar or pitch. In such extreme conditions, lumber should be at or below equilibrium moisture content at fabrication, to reduce subsequent shrinkage, which could open avenues of attack for the corrosive atmosphere.

Iron salts are frequently very acidic and show hydrolytic action on wood in the presence of free water. This accounts for softening and discoloration of wood observed around corroded nails. This action is especially pronounced in acidic woods, such as oak, and in woods containing considerable tannin and related compounds, such as redwood. It can be eliminated, however, by using zinc-coated aluminum, or copper nails.

11.16.1 Nails and Spikes

Common wire nails and spikes conform to the minimum sizes in Table 11.16.

Hardened deformed-shank nails and spikes are made of high-carbon-steel wire and are headed, pointed, annularly or helically threaded, and heat-treated and tempered, to provide greater strength than common wire nails and spikes. But the same loads are given for common wire nails and spikes or the corresponding lengths are used with a few exceptions.

Nails should not be driven closer together than half their length, unless driven in prebored holes. Nor should nails be closer to an edge than one-quarter their length. When one structural member is joined to another, penetration of nails into the second or farther timber should be at least half the length of the nails. Holes for nails, when

Table 11.16 Nail and Spike Dimensions

Pennyweight	Length, in	Wire Dia, in
Nails:		
6d	2	0.113
8d	2½	0.131
10d	3	0.148
12d	3¼	0.148
16d	3½	0.162
20d	4	0.192
30d	4½	0.207
40d	5	0.225
50d	5½	0.244
60d	6	0.263
Spikes:		
10d	3	0.192
12d	3¼	0.192
16d	3½	0.207
20d	4	0.225
30d	4½	0.244
40d	5	0.263
50d	5½	0.283
60d	6	0.283
5⁄16	7	0.312
⅜	8½	0.375

necessary to prevent splitting, should be bored with a diameter less than that of the nail. If this is done, the same allowable load as for the same-size fastener with a bored hole applies in both withdrawal and lateral resistance.

Nails or spikes should not be loaded in withdrawal from the end grain of wood. Also, nails inserted parallel to the grain should not be used to resist tensile stresses parallel to the grain.

Design values for nails and spikes and adjustment factors are discussed in Art. 11.17.

11.16.2 Wood Screws

The common types of wood screws have flat, oval, or round heads. The flathead screw is commonly used if a flush surface is desired. Oval- and round-headed screws are used for appearance or when countersinking is objectionable.

Wood screws should not be loaded in withdrawal from end grain. They should be inserted perpendicular to the grain by turning into predrilled holes and should not be started or driven with a hammer. Spacings, end distances, and side distances must be such as to prevent splitting.

For Douglas fir and southern pine, the lead hole for a screw loaded in withdrawal should have a diameter of about 70% of the root diameter of the screw. For lateral resistance, the part of the hole receiving the shank should be about seven-eighths the diameter of the screw at the root of the thread.

Design values for wood screws and adjustment factors are discussed in Art. 11.17.

11.16.3 Lag Screws

Also known as lag bolts, lag screws are large screws with a square or hexagonal bolt head. They range, usually, from about 0.2 to 1.0 in in diameter and from 1 to 16 in in length. The threaded portion ranges from ¾ in for 1- and 1¼-in-long lag screws to half the length for all lengths greater than 10 in.

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 particularly where it would be difficult to fasten a bolt or where a nut on the surface would be objectionable. They also are used instead of bolts where the components of a joint are so thick that an excessively long bolt would be needed or where heavy withdrawal loads have to be resisted.

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.

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. Penetration of the threaded portion to a distance of about 7 times the shank diameter in the denser species and 10 to 12 times the shank diameter in the less dense species will develop approximately the ultimate tensile strength of a lag screw.

Lag screws should preferably not be driven into end grain because splitting may develop under lateral load. The resistance of a lag screw to withdrawal from end grain is about three-quarters that from side grain.

Spacings, edge and end distances, and net section for lag-screw joints should be the same as those for joints with bolts of a diameter equal to the shank diameter of the lag screw.

For more than one lag screw, the total allowable load equals the sum of the loads permitted for each lag screw, provided that spacings, end distances, and edge distances are sufficient to develop the full strength of each lag screw.

Design values for lag screws and adjustment factors are discussed in Art. 11.17.

11.16.4 Bolts and Dowels

Machine bolts conforming to ANSI/ASME Standard B18.2.1, with square heads and nuts, are used extensively in wood construction. Spiral-shaped dowels are also used at times to hold two pieces of wood together; they are used to resist checking and splitting in railroad ties and other solid-sawn timbers.

Holes for bolts should always be prebored and have a diameter that permits the bolt to be driven easily (Art. 11.6). Careful centering of holes in main members and splice plates is necessary. The holes should have a diameter from $\frac{1}{32}$ to $\frac{1}{16}$ in larger than the bolt diameter. 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 11.8 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 11.17, for minimum edge distance in Table 11.18, and for minimum spacing between rows and between bolts in a row in Table 11.19. The geometry factor C_Δ discussed in Art. 11.17 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.

The critical section is that section at right angles to the direction of the load that gives maximum stress in the member over the net area remaining after bolt holes at the section are deducted. For parallel-to-grain loads, the net area at a critical section should be at least 100% for hardwoods and 80% for softwoods of the total area in bearing under all the bolts in the joint.

For parallel- or perpendicular-to-grain loads, spacing between rows paralleling a member should not exceed 5 in unless separate splice plates are used for each row.

Groups of Bolts ▪ When bolts are properly spaced and aligned, the allowable load on a group

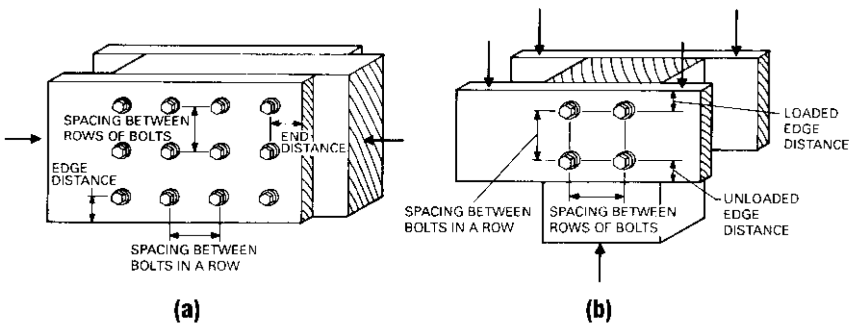

Fig. 11.8 Bolt spacing and edge distances in connections are defined with respect to load direction: (*a*) Parallel to grain; (*b*) perpendicular to grain. (*From F. S. Merritt and J. T. Ricketts, "Building Design and Construction Handbook," 5th ed., McGraw-Hill Publishing Company, New York.*)

Table 11.17 Minimum End Distance 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.

of bolts may be taken as the sum of the individual load capacities.

Design Values ▪ These and adjustment factors for bolts are discussed in Art. 11.17.

11.16.5 Timber Connectors

These are metal devices used with bolts for producing joints with fewer bolts 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 used with these connectors prevent the timbers from separating. The load is transmitted across the joint through the connectors.

Split rings, used for joining wood to wood, 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 (Fig. 11.9*b*). A bolt hole is drilled through the center of the core encircled by the groove. Split rings require greater accuracy for fabricating the wood members properly and the relative difficulty of installation make these connectors more costly than shear plates.

Shear plates are intended for wood-to-steel connections (Fig. 11.9*c* and *d*). But when used in pairs, they may be used for wood-to-wood connections (Fig. 11.9*e*), replacing split rings. 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 through each plate is to prevent the components of the joint from separating; loads are transmitted across the joint through the plates. They come in 2⅝- and 4-in diameters.

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. Shear plates are the prime connectors for timber construction subject to heavy loads.

Tables in the NDS list the least thickness of member that should be used with the various sizes of connectors. The NDS also lists minimum end and edge distances and spacing for timber connectors (Table 11.20). Edge distance is the distance from the edge of a member to the center

Table 11.18 Minimum Edge Distance for Bolts

Direction of loading*	Minimum edge distance
Parallel to grain:	
When $L/D \le 6$	$1.5D$
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.

Table 11.19 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.

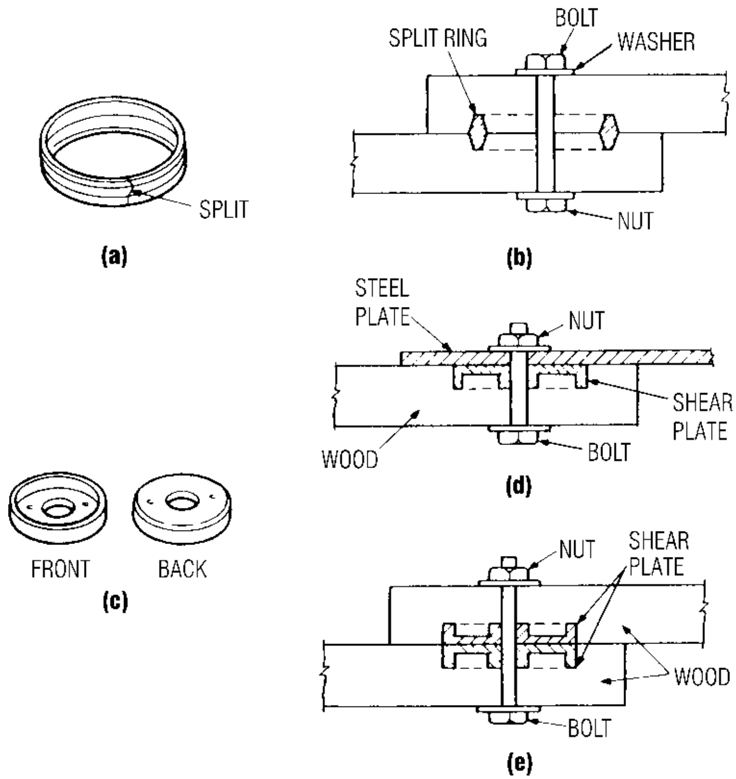

Fig. 11.9 Timber connectors: (*a*) Split ring; (*b*) wood members connected with split ring and bolt; (*c*) shear plate; (*d*) steel plate connected to a wood member with a shear plate and bolt; (*e*) wood members connected with a pair of shear plates and bolt.

Table 11.20 Minimum Edge and End Distances, Spacing, and Geometry Factors C_Δ for Shear-Plate Connectors.

	2⅝-in Shear-Plate Connectors				4-in Shear-Plate Connectors			
	Loads Parallel to Grain		Loads Perpendicular to Grain		Loads 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	1¾	1¾	1¾	1¾	2¾	2¾	2¾	2¾
C_Δ	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Loaded edge, in	1¾	1¾	1¾	2¾	2¾	2¾	2¾	3¾
C_Δ	1.0	1.0	0.83	1.0	1.0	1.0	0.83	1.0
End distance								
Tension member, in	2¾	5½	2¾	5½	3½	7	3½	7
C_Δ	0.625	1.0	0.625	1.0	0.625	1.0	0.625	1.0
Compression member, in	2½	4	2¾	5½	3½	5½	3½	7
C_Δ	0.625	1.0	0.625	1.0	0.625	1.0	0.625	1.0
Spacing								
Spacing parallel to grain, in	3½	6¾	3½	3½	5	9	5	5
C_Δ	0.5	1.0	1.0	1.0	0.5	1.0	1.0	1.0
Spacing perpendicular to grain, in	3½	3½	3½	4¼	5	5	5	6
C_Δ	1.0	1.0	0.5	1.0	1.0	1.0	0.5	1.0

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.

Placement of connectors in joints with members at right angles to each other is subject to the limitations of either member. Since rules for alignment, spacing, and edge and end distance of connectors for all conceivable directions of applied load would be complicated, designers must rely on a sense of proportion and adequacy in applying the above rules to conditions of loading outside the specific limitations mentioned.

Design values for shear plates and adjustment factors are discussed in Art. 11.17.

11.16.6 Anchor Bolts

To attach columns or arch bases to concrete foundations, anchor bolts are embedded in the concrete, with sufficient projection to permit placement of angles or shoes bolted to the wood. Sometimes, instead of anchor bolts, steel straps are embedded in the concrete with a portion projecting above for bolt attachment to the wood members.

11.16.7 Washers

Bolt heads and nuts bearing on wood require metal washers to protect the wood and to distribute the pressure across the surface of the wood. Washers may be cast, malleable, cut, round-plate, or square-plate. When subjected to salt air or salt water, they should be galvanized or given some type of effective coating. Ordinarily, washers are dipped in red lead and oil prior to installation.

Setscrews should never be used against a wood surface. It may be possible, with the aid of proper washers, to spread the load of the setscrew over sufficient surface area of the wood that the compression strength perpendicular to grain is not exceeded.

11.16.8 Tie Rods

To resist the horizontal thrust of arches not buttressed, tie rods are required. The tie rods may be installed at ceiling height or below the floor.

11.16.9 Hangers

Standard and special hangers are used extensively in timber construction. Stock hangers are available from a number of manufacturers. But by far the greater number of hangers are of special design. Where appearance is of prime importance, concealed hangers are frequently selected.

Design values for shear plates subject to loads at an angle between 0° (parallel to grain) and 90° (perpendicular to grain) may be computed from Eq. (11.21). In this case, F'_n, F'_g, and $F'_{c\perp}$ are, respectively, the adjusted design value at inclination θ with the direction of grain, parallel to grain, and perpendicular to grain.

Article 11.19 illustrates connections often used in wood structural framing.

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. The design values listed in the NDS 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.

11.17 Design Values and Adjustment Factors for Mechanical Fastenings

Determination of stress distribution in connections made with wood and metal is complicated. Consequently, 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 and Paper Association, lists 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.

11.17.1 Adjustment of Design Values for Connections with Fasteners

Nominal design values for connections or wood members with fasteners should be multiplied by applicable adjustment factors described in Art. 11.17.2 to obtain adjusted design values. The types of loading on the fasteners may be divided into four classes: lateral loading, withdrawal, loading parallel to grain, and loading perpendicular to grain. Adjusted design values are given in terms of nominal design values and adjustment factors in Eqs. (11.28) to (11.40),

where Z' = adjusted design value for lateral loading

Z = nominal design value for lateral loading

W' = adjusted design value for withdrawal

W = nominal design value for withdrawal

P' = adjusted value for loading parallel to grain

P = nominal value for loading parallel to grain

Q' = adjusted value for loading normal to grain

Q = nominal value for loading normal to grain

Bolts:

$$Z' = ZC_DC_MC_tC_gD_\Delta \tag{11.28}$$

where C_D = load-duration factor, not to exceed 1.6 for connections

C_M = wet-service factor, not applicable to toenails loaded in withdrawal

C_t = temperature factor

C_R = group-action factor

C_Δ = geometry factor

Split-ring and shear-plate connectors:

$$P' = PC_DC_MC_tC_gC_\Delta C_dC_{st} \tag{11.29}$$

$$Q' = QC_DC_MC_tC_gC_\Delta C_d \tag{11.30}$$

where C_d = penetration-depth-factor

C_{st} = metal-side-plate factor

Nails and spikes:

$$W' = WC_DC_MC_tC_{tn} \tag{11.31}$$

$$Z' = ZC_DC_MC_tC_dC_{eg}C_{di}C_{tn} \tag{11.32}$$

where C_{di} = diaphragm factor

C_{tn} = toenail factor

Wood screws:

$$W' = WC_DC_MC_t \tag{11.33}$$

$$Z' = ZC_DC_MC_tC_dC_{eg} \tag{11.34}$$

where C_{eg} = end-grain factor

Lag screws:

$$W' = WC_DC_MC_tC_{eg} \tag{11.35}$$

$$Z' = ZC_DC_MC_tC_gC_\Delta C_dC_{eg} \tag{11.36}$$

Metal plate connectors:

$$Z' = ZC_DC_MC_t \tag{11.37}$$

Drift bolts and drift pins:

$$W' = WC_DC_MC_tC_{eg} \tag{11.38}$$

$$Z' = ZC_DC_MC_tC_gC_\Delta C_dC_{eg} \tag{11.39}$$

Spike grids:

$$Z' = ZC_DC_MC_tC_\Delta \tag{11.40}$$

Adjustments for Fire-Retardant Treatment ▪ For connections made with lumber or structural glued-laminated timber 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.

11.17.2 Adjustment Factors for Connections with Fasteners

Design values for connections with fasteners should be adjusted as indicated in Art. 11.7.1. The adjustment factors are the following:

Load-Duration Factor ▪ Except when connection capacity is governed by the strength of metal, values of C_D may be taken from Table 11.5, Art. 11.4.2. For connections, C_D may not exceed 1.6.

Wet-Service Factor ▪ Nominal design values apply to wood that will be used 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 11.21

Temperature Factor ▪ Values of C_t are listed in Table 11.22 for connections that will experience sustained exposure to elevated temperatures from 100 to 150 °F.

Group-Action Factor ▪ Values of C_g are given in Table 11.23. The NDS contains design criteria for determination of C_g for configurations

Table 11.21 Wet-Service Factors C_M for Connections

Fastener Type	Conditions of Wood* At Time of Fabrication	In Service	C_M
Split-ring or shear-plate connectors†	Dry	Dry	1.0
	Partially seasoned	Dry	‡
	Wet	Dry	0.8
	Dry or wet	Partially seasoned or wet	0.67
Bolts or lag screws	Dry	Dry	1.0
	Partially seasoned or wet	Dry	§
	Dry or wet	Exposed to weather	0.75
	Dry or wet	Wet	0.67
Wood screws	Dry or wet	Dry	1.0
	Dry or wet	Exposed to weather	0.75
	Dry or wet	Wet	0.67
Common wire nails, box nails:			
For withdrawal loads	Dry	Dry	1.0
	Partially seasoned or wet	Wet	1.0
	Partially seasoned or wet	Dry	0.25
	Dry	Subject to wetting and drying	0.25
For lateral loads	Dry	Dry	1.0
	Partially seasoned or wet	Dry or wet	0.75
	Dry	Partially seasoned or wet	0.75

* Conditions of wood for determining wet-service factors for connections:
Dry wood—moisture content up to 19%.
Wet wood—moisture content at or above 30% (approximate fiber saturation point).
Partly seasoned wood—moisture content between 19% and 30%.
Exposed to weather—wood will vary in moisture content from dry to partly seasoned but is not expected to reach the fiber saturation point when the connection is supporting full design load. Subject to wetting and drying—wood will vary in moisture content from dry to partly seasoned or wet, or vice versa, with consequent effects on the tightness of the connection.

† For split-ring or shear-plate connectors, moisture-content limitations apply to a depth of ¾ in below the surface of the wood.

‡ When split-ring or shear-plate connectors are installed in wood that is partly seasoned at time of fabrication but will be dry before full design load is applied, proportional intermediate wet-service factors may be used.

§ When bolts or lag screws are installed in wood that is wet at the time of fabrication but will be dry before full design load is applied, the following wet service factors C_M apply:

For one fastener only or two or more fasteners placed in a single row parallel to grain or fasteners placed in two or more rows parallel to grain with separate splice plates for each row, $C_M = 1.0$.

When bolts or lag screws are installed in wood that is partly seasoned at the time of fabrication but will be dry before full design load is applied, proportional intermediate wet-service factors may be used.

not included in the table. For determination of C_g, a row of fasteners is defined as any of the following:

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.

The group-action factor is applied because the two end fasteners carry a larger load than the interior fasteners. With six or more fasteners in a row, the two end fasteners carry more than 50% of the load. With bolts, however, a small redistribution of load from the end bolts to the interior bolts occurs due to

Table 11.22 Temperature Factor C_t for Connections

In-Service Moisture Conditions*	$T \leq 100°F$	$100°F \leq T \leq 125°F$	$125°F < T \leq 150°F$
Dry	1.0	0.8	0.7
Wet	1.0	0.7	0.5

* Wet and dry service conditions are defined in a Table 11.21 footnote.

Table 11.23 Group-Action Factors

(*a*) For Bolt or Lag-Screw Connections with Wood Side Members*

		Number of Fasteners in a Row										
A_s/A_m †	A_s,‡ in^2	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§

		Number of Fasteners in a Row										
A_s/A_m †	A_s,‡ in^2	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.

† 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 $A_s/A_m > 1$, use A_m/A_s.

‡ When $A_s/A_m > 1$, 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 $2\frac{1}{2}$-in split-ring connectors, $2\frac{5}{8}$-in shear-plate connectors, $s < 9$ in, or $E > 1{,}400{,}000$ psi.

crushing of the wood at the end bolts. If failure is in shear, though, a partial failure occurs before substantial redistribution of load takes place.

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.

Geometry Factor ▪ When the end distance or the spacing is less than the minimum required by the NDS for full design value but larger than the minimum required for reduced design value for bolts, lag screws, and split-ring and shear-plate connectors, 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 (Table 11.20). 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.

Penetration Factor ▪ For wood screws, lag screws, nails, and spikes, when the penetration is larger than the minimum required by the NDS (Table 11.24) but less than that assumed in the establishment of the full lateral design value, linear interpolation should be used in determination of C_d. This factor should not exceed unity. Table 11.24 lists values of C_d for the aforementioned fasteners.

End-Grain Factor ▪ Application of C_{eg} is necessary because connections are weaker when fasteners, such as screws and nails, are inserted in the end grain than when they are inserted in the side grain. Woods screws, nails, and spikes should not be loaded in withdrawal from end grain. Lag screws may be so loaded, but the nominal design value should be multiplied by $C_{eg} = 0.75$. Wood screws, lag screws, nails, and spikes may be permitted to carry lateral loading when inserted, parallel to grain, into end grain. In such cases, the nominal design value for lateral loads should be multiplied by $C_{eg} = 0.67$.

Table 11.24 Penetration and 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 = bolt diameter.

Metal-Side-Plate Factor ▪ When metal side plates are used in joints made with nails, spikes, or wood screws, the design value for wood side plates may be multiplied by the metal-side-plate factor $C_{st} = 1.25$. 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 11.25. The values depend on the species of wood used in the connection, such as group A, B, C, or D listed in the NDS.

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 $C_{di} = 1.1$.

Toenail Factor ▪ For such connections as stud-to-plate, beam-to-plate, and blocking-to-plate nailing, 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 $C_{tn} = 0.83$.

Table 11.25 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.

11.18 Glued Joints

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). 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.

Only in special cases may lumber be glued with the grain direction of adjacent pieces at an angle. When the angle is large, dimensional changes from changes 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, but they are seldom strong enough 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. For this reason plane sloping scarfs of relatively flat slope (Fig. 11.2) or finger joints with thin tips and flat slope on the sides of the individual fingers (Fig. 11.3) are used 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 above 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 computed Eq. (11.21).

[Military Specification MIL-A-397B, "Adhesive, Room-Temperature and Intermediate-Temperature Setting Resin (Phenol, Resorcinol, and Melamine Base)," and Military Specification MIL A-5534A, "Adhesive, High-Temperature Setting Resin (Phenol, Melamine, and Resorcinol Base)," U.S. Naval Supply Depot, Philadelphia, PA 19120.]

11.19 Wood Structural Framing Details

Wood structural frames are frequently used for single-family residences, apartment buildings, and commercial and industrial structures. The framing usually is wood beam-and-girder with wood columns, wood beam-and-post, wood joists with wood-stud bearing walls, or glued-laminated timber arches or rigid frames. Roofs may be supported on wood trusses or sloping wood rafters.

Timber bridges generally are the trestle, girder, truss, or arch types. If sawn timbers are used, they should be pressure-treated with a preservative after fabrication. For glued-laminated members, either individual laminations should be pressure-treated with a preservative before they are glued together or the member should be treated after the gluing, depending on the type of treatment specified. Some preservative treatments may not be suitable for use after gluing. (Consult a local laminator or "Standard for Preservative Treatment of Structural Glued Laminated Timber," AITC 109, American Institute of Timber Construction, or both.) See also Art. 11.25.

Connections in the structural framing in buildings and bridges are made with mechanical fastenings, such as nails, spikes, wood screws, lag screws, bolts, and timber connectors (see Arts. 11.16 and 11.17). Standard and special pre-engineered metal hangers are used extensively. Stock hangers are generally available and most manufacturers also provide hangers of special design. Where appearance is of prime importance, concealed hangers may be specified.

Figures 11.10 to 11.12 show structural framing details such as beam hangers and connectors and column anchors.

Wood Framing for Small Houses ▪ Although skeleton framing may be used for one- and two-family dwellings, such structures up to three stories high generally are built with load-bearing walls. When wood framing is used, the walls are conventionally built with slender studs spaced 16 or 24 in c to c. Similarly, joists and

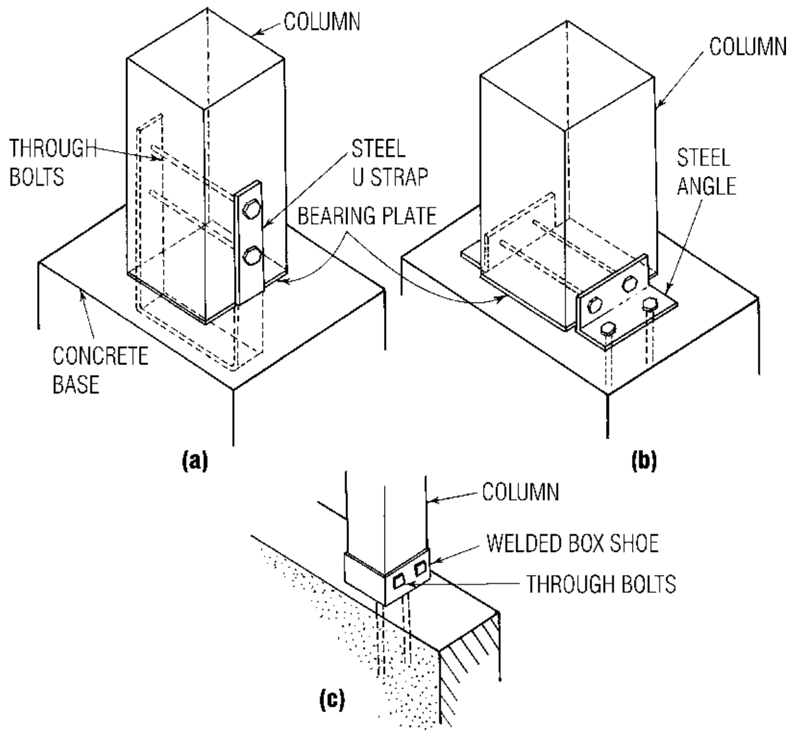

Fig. 11.10 Typical anchorages of wood column to base: (*a*) Wood column anchored to concrete base with U strap; (*b*) anchorage with steel angles; (*c*) with a welded box shoe.

rafters, which are supported on the walls and partitions, are usually also spaced 16 or 24 in c to c. Facings, such as sheathing and wallboard, and decking, floor underlayment, and roof sheathing are generally available in appropriate sizes for attachment to studs, joists, and rafters with that spacing.

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 plank, called the bottom or sole plate, and at the top to a pair of horizontal planks, 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, called a ribband, supported in a cutout in the studs (Fig. 11.14).

Studs may be braced against racking by diagonals or horizontal blocking and facing materials, such as plywood or gypsum sheathing.

Three types of wood-frame construction are generally used: platform frame, balloon frame, and plank-and-beam frame.

In **platform framing**, first-floor joists are completely covered with subflooring to form a platform on which exterior walls and interior partitions are built (Fig. 11.13). This is the type of framing usually used for single-family dwellings.

Balloon framing is generally used for construction more than one story high. Wall studs are continuous from story to story. First-floor joists and exterior wall studs bear on an anchored sill plate (Fig. 11.14). Joists for the second and higher floors bear on a 1×4-in ribband let in to the inside edges of exterior wall studs. In two-story buildings, with brick or stone veneer exteriors, balloon framing minimizes variations in settlement of the framing and the masonry veneer.

Plank-and-beam framing (Fig. 11.15) requires fewer but larger-size piers, and wood components are spaced farther apart than in platform and balloon framing. In plank-and-beam framing, subfloors or roofs, typically composed of planks with a nominal thickness of 2 in, are supported on beams

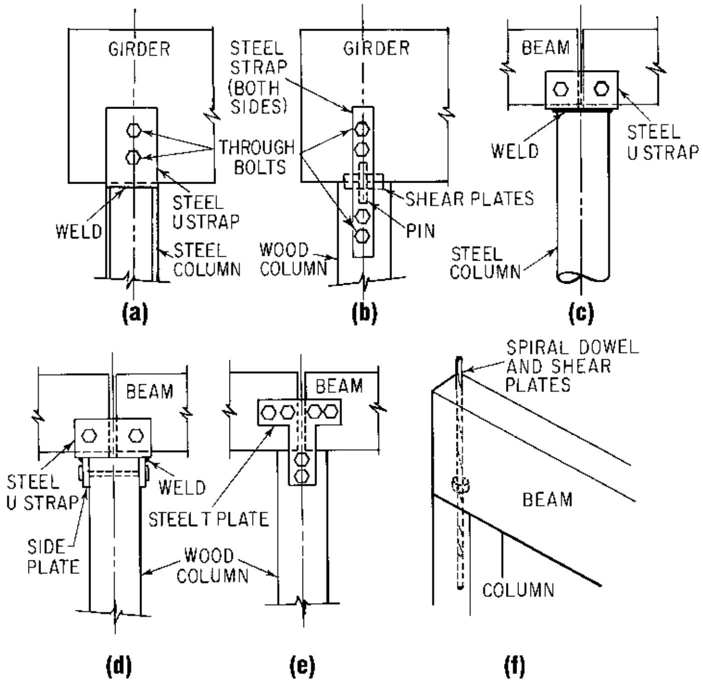

Fig. 11.11 Typical wood beam and girder connections to columns: (*a*) Wood girder to steel column; (*b*) girder to wood column; (*c*) beam to steel pipe column; (*d*) beam to wood column, with steel strap welded to steel side plates; (*e*) beam to wood column, with a steel T plate; (*f*) beam to wood column with a spiral dowel and shear plates.

spaced 8 ft c to c. Ends of the beams are supported on posts or concrete piers. Supplemental framing, set between posts for attachment of exterior and interior wall framing and finishes, also provides lateral support or bracing for the frame. Construction labor savings result from the use of larger and fewer framing members, which require less handling and fewer mechanical fasteners. Another advantage is that the need for cross bracing, which is often required in platform and balloon framing, is eliminated.

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

11.20 Design of Wood Trusses

Wood trusses are used for long-span bridges and for support of roofs for buildings. For the latter, trusses offer the advantage that the type and arrangement of members may be chosen to suit the shape of the structure and the loads and stresses involved. Prefabricated, lightweight wood and wood-steel trusses are available and offer economy through use of repetitive design and mass production in truss assembly plants.

Joints are critical in truss design. Use of a specific truss type is often governed by joint considerations.

11.20.1 Lightweight Trusses

Chords and web members of lightweight trusses are generally made of dimension lumber, either visually graded or machine stress-rated. The trusses are usually installed 12 to 24 in c to c and are designed to take advantage of repetitive-member action (Art. 11.4.9). At a joint, members are connected by sheet-metal-gusset nail plates with projections, or teeth, that are pressed into the wood on opposite faces of the joint.

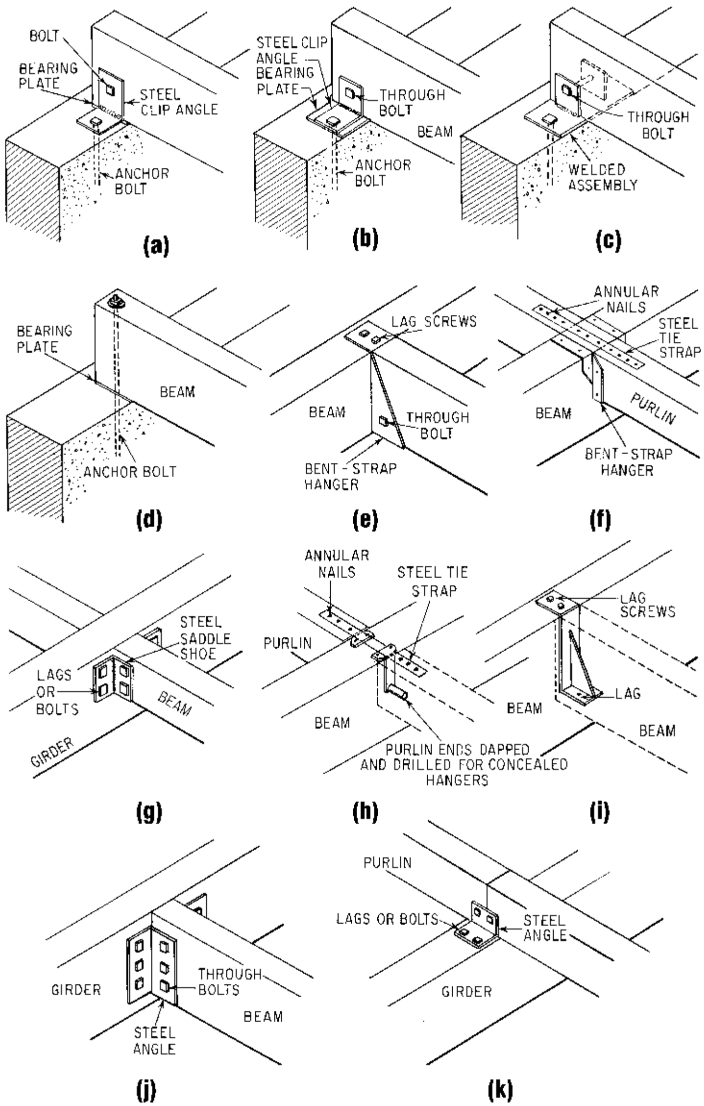

Fig. 11.12 Beam connections: (*a*) and (*b*) Wood beam anchored on a wall with steel angles; (*c*) with welded assembly; (*d*) beam anchored directly with a bolt; (*e*) beam supported on a bent-strap hanger on a girder; (*f*) similar support for purlins; (*g*) saddle connects beam to girder (suitable for one-sided connection); (*h*) and (*i*) connections with concealed hangers; (*j*) and (*k*) connections with steel angles.

Load-transfer capacity at a joint is based on an allowable load per unit of surface area of plate. Accordingly, a plate should be sized to cover all the members at the joint with an area sufficient to transfer loads from each member to the others. The allowable load depends on the number, size, and design of the steel teeth of the gusset plate. The load capacities of specific gusset plates should be obtained from the manufacturer. Additional information on this type of truss may be obtained from the Truss Plate Institute and the Wood Truss Council of America, both located in Madison, Wisconsin.

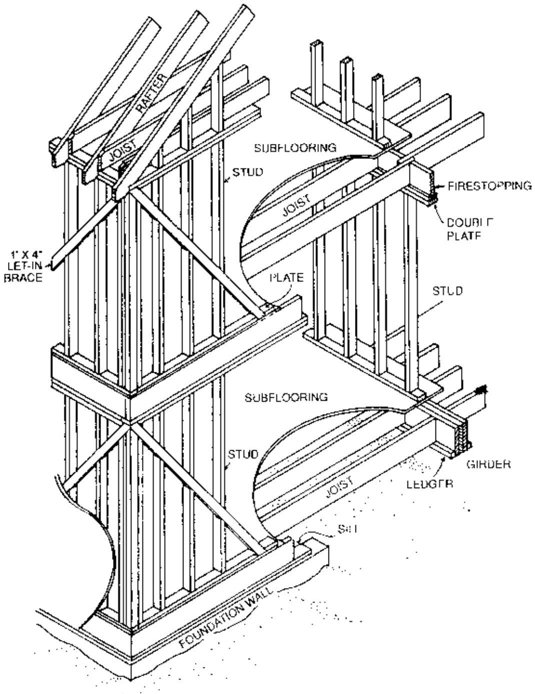

Fig. 11.13 Platform framing for two-story building.

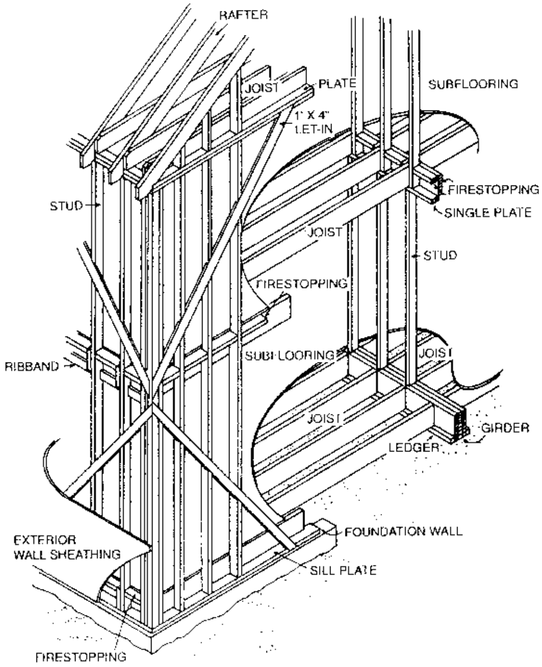

Fig. 11.14 Balloon framing for two-story building.

11.20.2 Timber Trusses

For long spans or large truss spacings, for example, 8 ft c to c or more, heavier wood chords and webs will be required. These members may have a nominal thickness of 3 or 4 in, or they may be glued-laminated timbers. At joints, the members will be connected with thicker metal-gusset plates than those required for lightweight trusses. As an alternative, composite wood-steel trusses with lumber chords and steel webs may be used.

Types of timber trusses generally used are bowstring, flat or parallel chord, and scissors (Fig. 11.16). For commercial buildings, trusses usually are spaced 8 to 24 ft apart.

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 the most common arrangements. Web members may be attached to the sides of the chords, or the 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. Glued-laminated chords and solid-sawn web members are usually used. Steel rods or other steel shapes may be used as members of timber trusses if they meet design and service requirements.

The bowstring truss is by far the most popular. In building construction, spans of 100 to 200 ft are common, with single or two-piece top and bottom chords of glued-laminated timber, webs of solid-sawn timber, and metal heel plates, chord splice plates, and web-to-chord connections. This system is light in weight for the loads that it can carry; it can be shop- or field-assembled. Attention to 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; web stresses are low under uniformly distributed loads.

Parallel-chord trusses, with slightly sloping top chords and level bottom chords, are used less often because chord stresses are not uniform along their length and web stresses are high. Hence, different

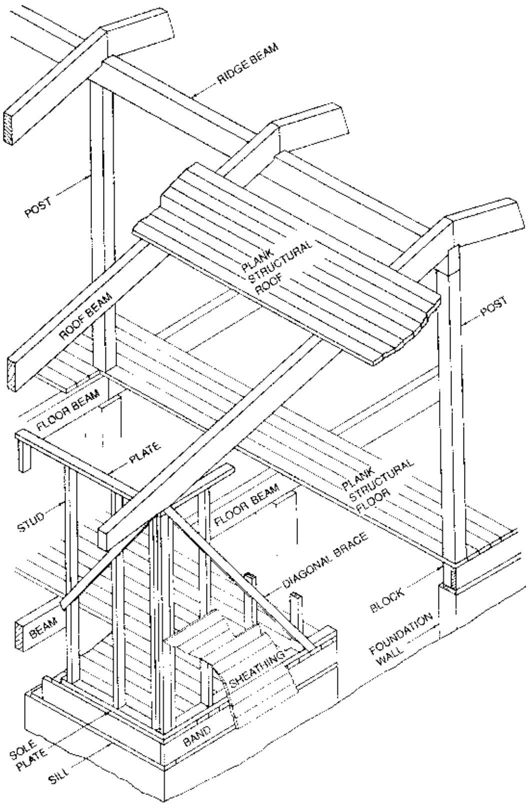

Fig. 11.15 Plank-and-beam framing for one-story building.

cross sections are required for successive chords, and web members and web-to-chord connections are heavy. Eccentric joints and tension stresses across the grain should be avoided in truss construction whenever possible, but particularly in parallel-chord trusses.

Triangular trusses and the more ornamental camelback and scissors trusses are used for shorter spans. They usually have solid-sawn members for both chords and webs where degree of seasoning of timbers, hardware, and connections are of considerable importance.

Truss Joints ▪ For joints, bolts, lag screws, metal-gusset nail plates (Art. 11.20.1), or shear-plate connectors are generally used. Sometimes, when small trusses are field-fabricated, only bolted joints are used. However, grooving tools for connectors can also be used effectively in the field. Metal-gusset plates are usually installed in a truss assembly plant.

Framing between Trusses ▪ Longitudinal sway bracing perpendicular to 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 consists of struts between trusses at bottom-chord level and diagonal tie rods, often of steel with turnbuckles for adjustment.

(“Design Manual for TECO Timber Connector Construction,” Timber Engineering Co., Colliers, W. Va.; AITC 102, app. A, “Trusses and Bracing,” American Institute of Timber Construction, Englewood, CO 80110; K. F. Faherty and T. G. Williamson, “Wood Engineering and Construction Handbook,” 2nd ed., McGraw-Hill Publishing Company, New York.)

11.21 Design of Timber Arches

Arches may be two-hinged, with hinges at each base, or three-hinged, with a hinge at the crown. Figure 11.17 presents typical forms of arches.

Tudor arches are gabled rigid frames with curved haunches. Columns and pitched roof beam on each side of the crown usually are one piece of glued-laminated timber. This type of arch is frequently used in church construction with a high rise.

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,

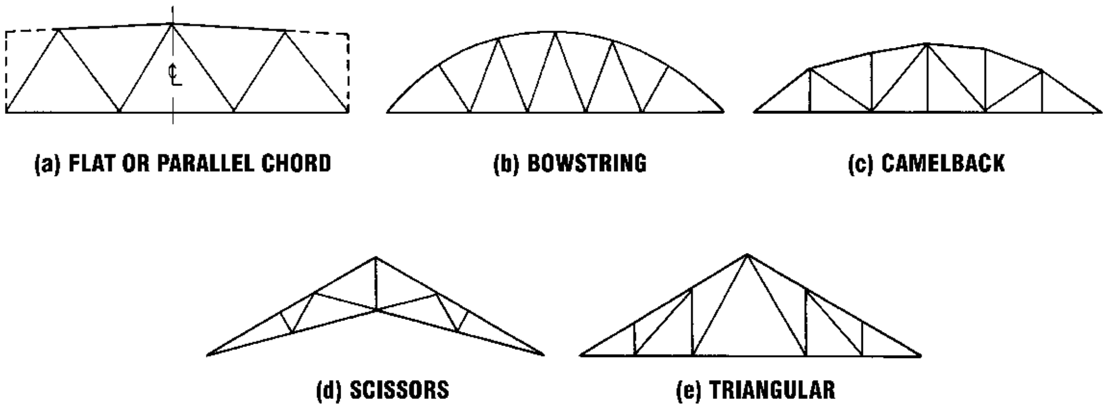

Fig. 11.16 Types of wood trusses.

and simple connections may be used where the arch is supported on masonry walls, concrete piers, or columns (Fig. 11.18).

Segmented arches are fabricated with overlapping lumber segments, nailed- or glued-laminated. They generally are three-hinged, and they may be tied or buttressed. They are economical because of the ease of fabrication and simplicity of field erection. Field splice joints are minimized; generally there is only one simple connection, at the crown (Fig. 11.19*c*). 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. And the cross section is large enough for segmented arches to be classified as heavy-timber construction.

A long-span arch may require a splice or moment connection to segment the arch to facilitate transportation to the jobsite. Figure 11.20 shows typical moment connections for wood arches.

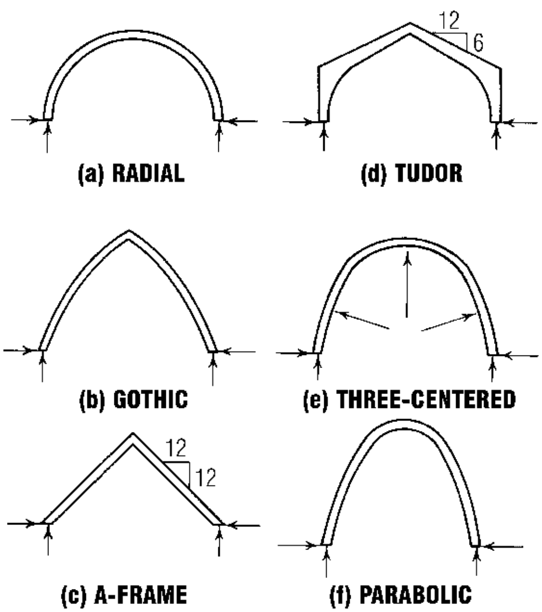

Fig. 11.17 Types of wood arches.

11.22 Timber Decking

Wood decking used for floor and roof construction may consist of solid-sawn planks with nominal thickness of 2, 3, or 4 in. Or it may be panelized or laminated. Panelized decking is made up of splined panels, usually about 2 ft wide.

For glued-laminated decking, two or more pieces of lumber are laminated into a single decking member, usually with 2- to 4-in nominal thickness.

Solid-sawn decking usually is fabricated with edges tongued and grooved or shiplapped for transfer of vertical load between pieces. The decking may be end matched, square end, or end grooved for splines. As indicated in Fig. 11.21, the decking may be arranged in various patterns over supports.

In Type 1, the pieces are simply supported. Type 2 has a controlled random layup. Type 3 contains intermixed cantilevers. Type 4 consists of a

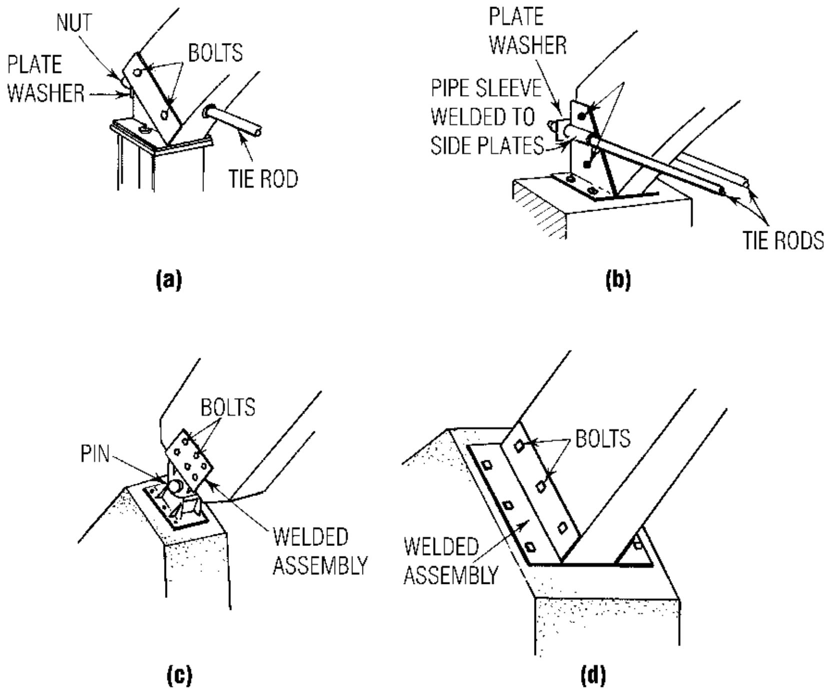

Fig. 11.18 Bases for segmented wood arches: (*a*) and (*b*) Tie rod anchored to arch shoe; (*c*) hinge anchorage for large arch; (*d*) welded arch shoe.

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 layup, 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 stress 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 (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. Each end at a support should be nailed to it. For 2-in decking, a $3\frac{1}{2}$-in (16d) toe and face nail should be used in each 6-in-wide piece at supports, and three nails for wider pieces. Tongue-and-groove decking generally is also toenailed through the tongue. For 3-in

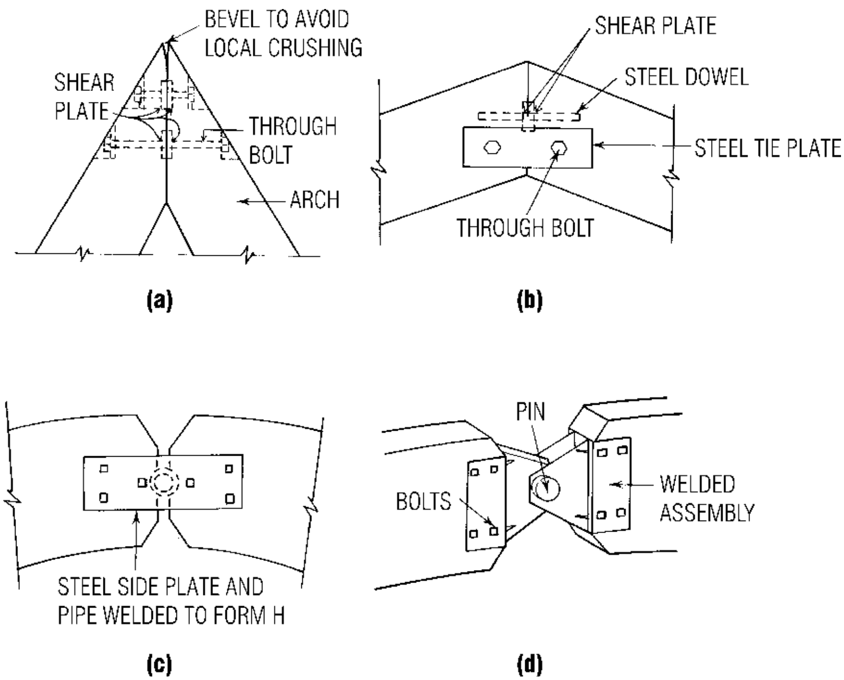

Fig. 11.19 Crown connections for arches: (*a*) For arches with slope 4 : 12 or greater, the connection consists of pairs of back-to-back shear plates with through bolts or threaded rods counterbored into the arch. (*b*) For arches with flatter slopes, shear plates centered on a dowel may be used in conjunction with the plates and through bolts. (*c*) and (*d*) Hinge at crown.

decking, each piece should be toenailed with one 4-in (20d) spike and face-nailed with 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.

(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, 7012 S. Revere Parkway, Englewood, Colo (www.AITC-glulam.org); AITC "Timber Construction Manual," 4th ed., John Wiley & Sons, Inc., New York (www.wiley.com).)

11.23 Pole Construction

Wood poles are used for various types of construction, including flagpoles, utility poles, and framing for buildings. These employ preservative-treated round poles set into the ground as columns. The ground furnishes vertical and horizontal support and prevents rotation at the base.

For allowable foundation and lateral pressures, consult the local building code or a model code.

In buildings, a bracing system can be provided at the top of the poles to reduce bending moments at the base and to distribute loads. Design of buildings supported by poles without bracing requires good knowledge of soil conditions, to eliminate excessive deflection or sidesway.

Bearing values under the base of poles should be checked. For backfilling the holes, well-tamped

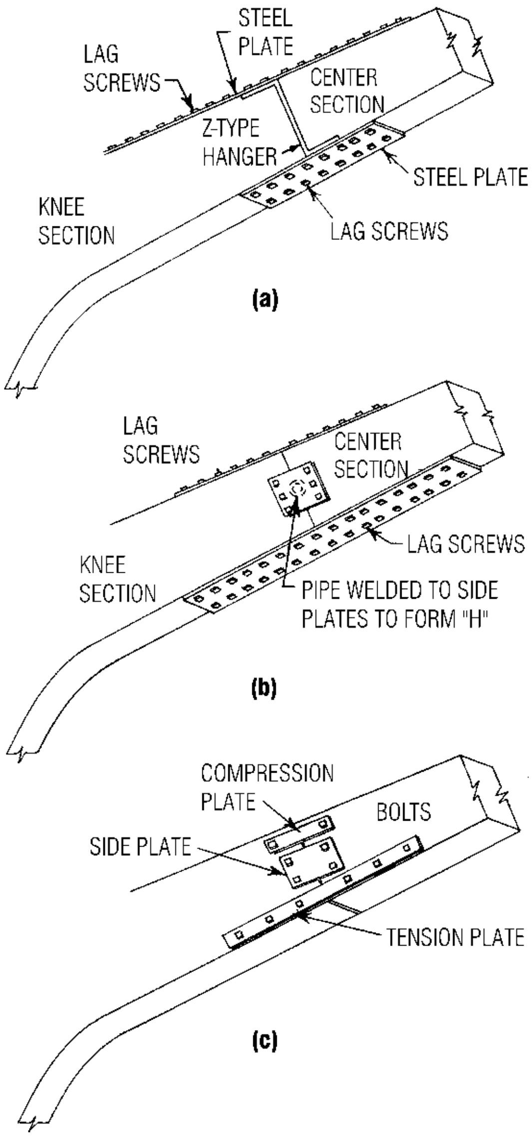

Fig. 11.20 Schematics of some moment connections for timber arches: (*a*) and (*b*) Connections with top and bottom steel plates; (*c*) connection with side plates.

native soil, sand, or gravel may be satisfactory. But concrete or soil cement is more effective. They can reduce the required depth of embedment and improve bearing capacity by increasing the skin-friction area of the pole. Skin friction is effective in reducing uplift due to wind.

To increase bearing capacity under the base end of poles for buildings, concrete footings often are used. They should be designed to withstand the punching shear of the poles 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.

Calculation of required depth of embedment in soil of poles subject to lateral loads generally is impractical without many simplifying assumptions. An approximate analysis can be made, but the depth of embedment should be checked by tests or at least against experience in the same type of soil. See "Post and Pole Foundation Design," ASAE Engineering Practice, EP486, American Society of Agricultural Engineers, St. Joseph, Mich.

("Design Properties of Round, Sawn and Laminated Preservatively Treated Construction Poles and Posts," ASAE Engineering Practice, EP388.2; "Standard Specifications and Dimensions for Wood Poles." ANSI 05.1, American National Standards Institute (www.ansi.org).)

11.24 Wood Structural Panels

Structural panels are composed of two or more materials with different structural characteristics formed into a thin, flat configuration and capable of resisting applied loads. The panels may be classified, in accordance with the manufacturing process, as plywood; mat-formed panels, such as oriented-strand board (OSB); and composite panels.

Plywood is a structural panel comprising wood veneer plies, united under pressure by adhesive. The bond between plies is at least as strong as solid wood. The panel is formed of an odd number of layers, with the grain of each layer perpendicular to the grain of adjoining layers. A layer may consist of a single ply or two or more plies laminated with grain parallel. Outer layers and all odd-numbered layers usually have the grain oriented parallel to the long dimension of the panel. The variation in grain direction, or cross lamination, makes the panel strong and stiff, equalizes strains under load, and limits panel dimensional changes, warping, and splitting.

Mat-formed panels are structural panels, such as particleboard, waferboard, and OSB, that do not contain wood veneer. Particleboard consists of a combination of wood particles and adhesive and is

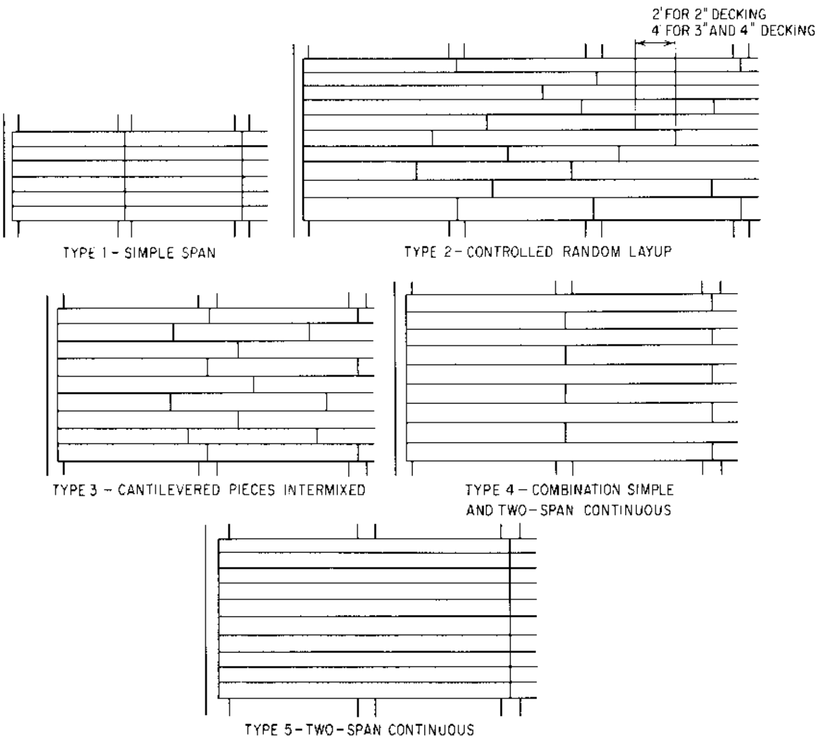

Fig. 11.21 Typical arrangements for heavy-timber decking.

widely used as floor underlayment in buildings. Waferboard is similar to particleboard but is made with flakes instead of particles. OSB is composed of compressed wood strands arranged in layers at right angles to one another and bonded with a waterproof adhesive. Like plywood, OSB has the strength and stiffness that result from cross lamination of layers.

Composite panels consist of combinations of veneer and other wood-based materials.

Structural wood panels may be used in construction as sheathing, decking, floor underlayment, siding, and concrete forms. Plywood, in addition, may serve as a component of stressed-skin panels and built-up (I- or box-shape) beams and columns.

To satisfy building code requirements, structural wood panels should meet the requirements of one or more of the following standards:

"U.S. Product Standard PS 1-83 for Construction and Industrial Plywood," applicable to plywood only.

"Voluntary Product Standard PS 2-92, Performance Standard for Wood-Based Structural-Use Panels," applicable to plywood, OSB, and composite panels.

"APA Performance Standards and Policies for Structural-Use Panels," PRP 108, which is similar to PS 2 but also incorporates performance-based qualifications procedures for siding panels.

11.24.1 Classification of Structural Panels

To meet building code requirements, structural wood panels should carry the trademark of a code-approved agency, such as the American Plywood Association (APA). Construction grades are generally fabricated with a waterproof adhesive and may be classified as Exterior or Exposure 1.

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

Exposure 1 panels may be used where they are not permanently exposed to the weather and where exposure durability is needed to resist the effects of moisture during construction delays, high humidity, water leakage, and other conditions of similar severity.

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

Interior panels are intended for interior use where they will be exposed only to minor amounts of moisture and only temporarily.

11.24.2 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 1-83.

Group 1. Douglas fir from Washington, Oregon, California, Idaho, Montana, Wyoming, British Columbia, and Alberta; western larch; southern pine (loblolly, longleaf, shortleaf, slash); yellow birch; tan oak

Group 2. Port Orford cedar; Douglas fir from Nevada, Utah, Colorado, Arizona, and New Mexico; fir (California red, grand, noble, Pacific silver, white); western hemlock; red and white lauan; western white pine; red pine; black maple; yellow poplar; red and Sitka spruce

Group 3. Red alder; Alaska cedar; jack, lodgepole, spruce and ponderosa pine; paper birch; subalpine fir; eastern hemlock; bigleaf maple; redwood; black, Engelmann, and white spruce

Group 4. Incense and western red cedar, sugar and eastern white pine, eastern and black (western poplar) cottonwood, cativo, paper birch, and bigtooth and quaking aspen

Group 5. Balsam fir, basswood, and balsam poplar

The 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 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.

11.24.3 Grades of Structural Wood Panels

Wood veneers are graded in accordance with appearance. Veneer grades define veneer appearance in terms of natural, unrepaired-growth characteristics and allowable number and size of repairs that may be made during manufacture (Table 11.26). 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.

Plywood is generally graded in accordance with 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. Since OSB panels are composed of flakes or strands instead of veneers, they are graded without reference to veneers or species. 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. 11.22.

Plywood panels with B-grade or better veneer faces are supplied in sanded-smooth condition 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.

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 28 ft.

Construction plywood is graded under the standard in accordance with two basic systems. One system covers engineered grades, the other appearance grades.

Engineered grades consist largely of unsanded sheathing panels designated C-D Interior or C-C Exterior. The latter is bonded with exterior glue. Either grade may be classified as Structural I or

Table 11.26 Veneer-Grade Designations

Grade 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.
Grade 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.
Grade B
Solid surface. Shims, circular repair plugs, and tight knots up to 1 in across grain permitted. Some minor splits and synthetic repairs permitted.
Grade C—Plugged
Improved C veneer with splits limited to ⅛-in width and knotholes and borer holes limited to ¼ × ½ in. Admits some broken grain. Synthetic repairs permitted.
Grade C
Tight knots to 1½ in. Knotholes up to 1 in across grain and some up to 1½ 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.
Grade D
Knots and knotholes up to 2½ in wide across grain and ½ in larger within specified limits, limited splits, and stitching permitted. Limited to Exposure 1 or interior panels.

Structural II, both of which are made with exterior glue and subject to other requirements, such as limitations on knot size and repairs of defects. Structural I is made only of Group I species and is stiffer than other grades. Structural II may be made of Group 1, 2, or 3 or any combination of these species. Structural I and II are suitable for such applications as box beams, gusset plates, stressed-skin panels, and folded-plate roofs.

Appearance grades, except for Plyform, are designated by panel thickness, veneer classification of face and back veneers, and species group of the veneers. For Plyform, class designates a mix of species.

11.24.4 Plywood Applications

Table 11.27 describes the various grades of plywood and indicates how they are generally used.

PS 1-83 classifies plywood made for use as concrete forms in two grades. Plyform (B-B) Class I is limited to Group I species on face and back, with limitations on inner plies. Plyform (B-B) Class II permits Group 1, 2, or 3 for face and back, with limitations on inner plies. High-density overlay should be specified for both classes when highly smooth, grain-free concrete surfaces and maximum reuses are required. The bending strength of Plyform Class I is greater than that of Class II. Grades other than Plyform, however, may be used for forms.

Span-rated panels are available that are designed specifically for use in buildings 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

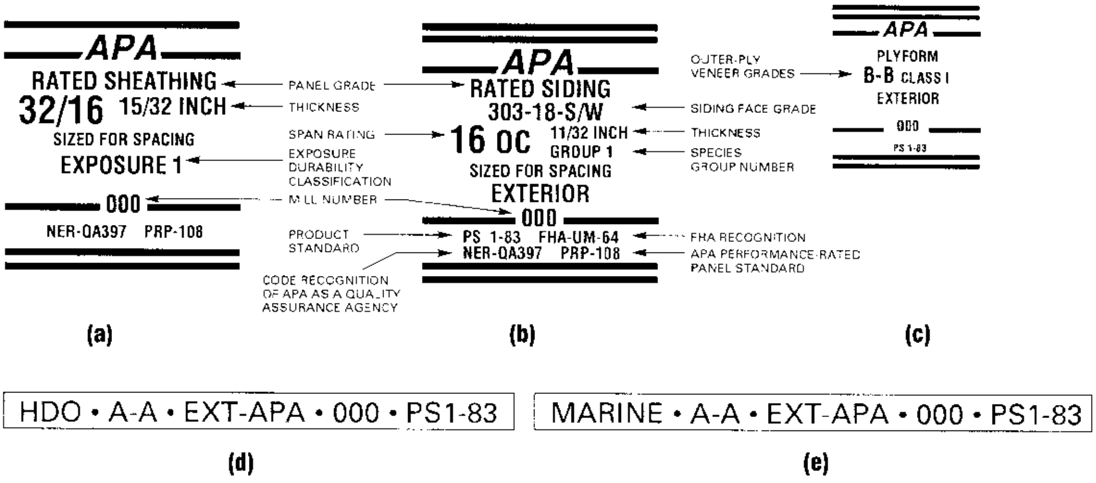

Fig. 11.22 Typical trademarks for structural panels. (*a*) APA Rated Sheathing with a thickness of $\frac{15}{32}$ in and a span rating $\frac{32}{16}$. The left-hand number denotes the recommended maximum spacing of supports when the panel is used for roof sheathing with the long dimension or strength 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. (*b*) APA Rated Siding, grade 303-18-S/W, with a span rating of 16 in. (*c*) APA Ply-form, 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.

strength axis across supports (Fig. 11.23*a*). 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 preferred, though panels may be nailed only. Figure 11.23*b* illustrates application of panel subflooring.

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, but most may also be placed horizontally (long dimension across supports) if horizontal joints are blocked.

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.

Rated sheathing meets building code wall-sheathing requirements for bending and racking strength without let-in corner bracing. Installation is illustrated in Fig. 11.24. Either rated sheathing or all-veneer plywood rated siding can be used in shear walls.

Publications of the American Plywood Association, P.O. Box 11700, Tacoma, WA 98411-0700 (www.apawood.org): “U.S. Product Standard PS 1-83 for Construction and Industrial Plywood,” H850; “Voluntary Product Standard PS 2-92,” S350; “Performance Standards and Policies for Structural-Use Panels,” E445; “Nonresidential Roof Systems,” A310; “APA Design Construction Guide, Residential & Commercial,” E30; “Diaphragms,” L350; “Concrete Forming,” V345; “Plywood Design Specifications (PDS)”, Y510; PDS “Supplements;” “House Building Basics,” X461.)

Table 11.27 Applications of Plywood Grades

Plywood Grade	Description and use	Veneer Grade			Common Thicknesses, in
		Face	Back	Inner	
(a) Interior-Type Plywood					
C-D INT-APA	Unsanded sheathing grade for wall, roof subflooring, and industrial applications such as pallets and for engineering design, with proper stresses. Also available with intermediate and exterior glue.* For permanent exposure to weather or moisture only, exterior-type plywood is suitable.	C	D	D	$\frac{5}{16}$, $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$
Structural I C-D INT-APA or Structural II C-D INT-APA	Plywood grades to use where strength properties are of maximum importance, such as plywood-lumber components. Made with exterior glue only. Structural I is made from all Group 1 woods. Structural II allows Group 3 woods.	C	D	D	$\frac{5}{16}$, $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$
Underlayment INT-APA	For underlayment or combination subfloor-underlayment under resilient floor coverings. Available with exterior glue. Touch-sanded. Available with tongue and groove.	C Plugged	D	C & D	$\frac{1}{2}$, $\frac{19}{32}$, $\frac{5}{8}$, $\frac{23}{32}$, $\frac{3}{4}$
C-D Plugged INT-APA	For built-ins, wall and ceiling tile backing, not for underlayment. Available with exterior glue. Touch-sanded.	C Plugged	D	D	$\frac{1}{2}$, $\frac{19}{32}$, $\frac{5}{8}$, $\frac{23}{32}$, $\frac{3}{4}$
Structural I or II† Underlayment or C-D plugged	For higher strength requirements for underlayment or built-ins. Structural I constructed from all Group 1 woods. Made with exterior glue only.	C Plugged	D	C & D	$\frac{1}{2}$, $\frac{19}{32}$, $\frac{5}{8}$, $\frac{23}{32}$, $\frac{3}{4}$
2.4.1 INT-APA	Combination subfloor-underlayment. Quality floor base. Available with exterior glue, most often touch-sanded. Available with tongue and groove.	C Plugged	D	C & D	$1\frac{1}{8}$
Appearance grades	Generally applied where a high quality surface is required. Includes N-N, N-A, N-B, N-D, A-A, A-B, A-D, B-B, and B-D INT-APA grades.	B or better	D or better	D	$\frac{1}{4}$, $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$

(Continued)

Table 11.27 (*Continued*)

Plywood Grade	Description and use	Veneer Grade Face	Veneer Grade Back	Veneer Grade Inner	Common Thicknesses, in
(*b*) Exterior-Type Plywood					
C-D EXT-APA	Unsanded sheathing grade with waterproof glue bond for wall, roof, subfloor, and industrial applications such as pallet bins.	C	C	C	$\frac{5}{16}$, $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$
Structural I C-C EXT-APA or Structural II C-C EXT-APA†	"Structural" is a modifier for this unsanded sheathing grade. For engineering applications in construction and industry where full exterior-type panels are required. Structural I is made from Group 1 woods only.	C	C	C	$\frac{5}{16}$, $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$
Underlayment EXT-APA and C-C plugged EXT-APA	Underlayment for combination subfloor underlayment or two-layer floor under resilient floor coverings where severe moisture conditions may exist. Also for controlled-atmosphere rooms and many industrial applications. Touch-sanded. Available with tongue and groove.	C Plugged	C	C	$\frac{1}{2}$, $\frac{19}{32}$, $\frac{5}{8}$, $\frac{23}{32}$, $\frac{3}{4}$
Structural I or II† Underlayment EXT-APA or C-C plugged EXT-APA	For higher-strength underlayment where severe moisture conditions may exist. All Group 1 construction in Structural I. Structural II allows Group 3 woods.	C Plugged	C	C	$\frac{1}{2}$, $\frac{19}{32}$, $\frac{5}{8}$, $\frac{23}{32}$, $\frac{3}{4}$
B-B Plyform Class I or II†	Concrete-form grade with high reuse factor. Sanded both sides, mill-oiled unless otherwise specified. Available in HDO. For refined design information on this special-use panel see APA publication "Plywood for Concrete Forming" (form V345). Design using values from this specification will result in a conservative design.	B	B	C	$\frac{5}{8}$, $\frac{3}{4}$
Marine EXT-APA	Superior exterior-type plywood made only with Douglas fir or Western larch. Special solid-core construction. Available with MDO or HDO face. Ideal for boat hull construction.	A or B	A or B	B	$\frac{1}{4}$, $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$

Table 11.27 (*Continued*)

Plywood Grade	Description and use	Veneer Grade			Common Thicknesses, in
		Face	Back	Inner	
Appearance grades	Generally applied where a high quality surface is required. Includes A-A, A-B, A-C, B-B, B-C, HDO, and MDO EXT-APA. Appearance grades may be modified to Structural I. For such designation use Group 1 stresses and Table 11-33*b* (sanded) section properties.	B or better	C or better	C	$\frac{1}{4}$, $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$

* When exterior glue is specified, i.e., "interior with exterior glue glue," stress level 2 (S-2) should be used.
† Check local suppliers for availability of Structural II and Plyform Class II grades.
Source: "Plywood Design Specifications," American Plywood Association.

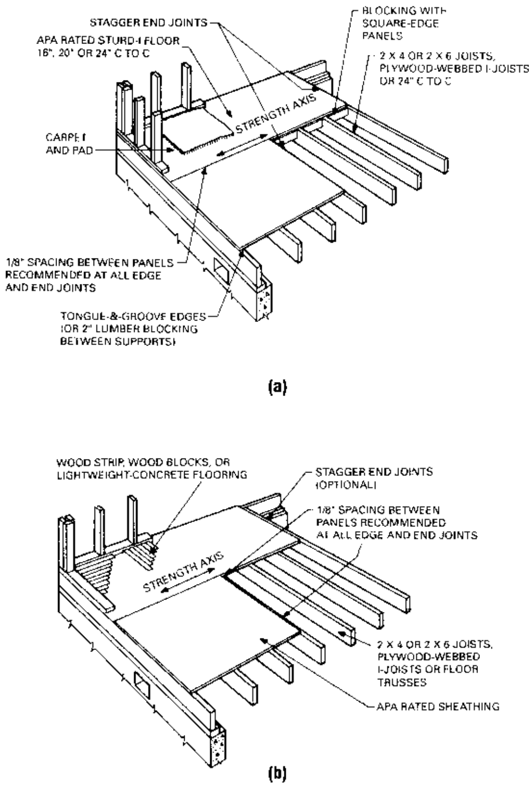

Fig. 11.23 Floor construction with structural wood panels: (*a*) Single-layer floor; (*b*) subfloor.

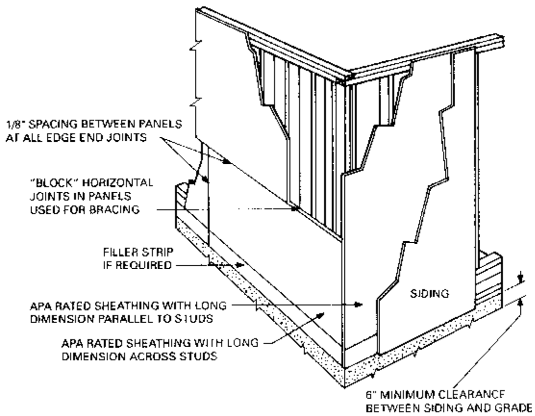

Fig. 11.24 Structural panel sheathing applied to studs.

11.25 Preservative Treatments for Wood

Wood-destroying fungi must have air, suitable moisture, and favorable temperatures to develop and grow in wood. Submerge wood permanently and totally in water to exclude air, keep the moisture content below 18 to 20%, or hold temperature below 40 °F or above 110 °F, and wood remains permanently sound. If wood moisture content is kept below the fiber-saturation point

(25 to 30%) when the wood is untreated, decay is greatly retarded. Below 18 to 20% moisture content, decay is completely inhibited.

If wood cannot be kept dry, a wood preservative, properly applied, must be used. The following can be a guide to determine if treatment is necessary.

Wood members are permanent without 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%. Also, in arid or semiarid regions, where climatic conditions are such that the equilibrium moisture content seldom exceeds 20%, and then only for short periods, wood members are permanent without treatment.

Where wood is in contact with the ground or 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 20%, wood structural members must be treated with a preservative, as must wood exposed outdoors without protective roof covering and where the wood moisture content can go above 18 to 20% for repeated or prolonged periods.

Where wood structural members are subject to condensation by being in contact with masonry, preservative treatment is 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, Stevensville, Md., describes these approved processes.) Design values for pressure-preservative-treated lumber are modified with the usual adjustment factors described in Art. 11.4 with one exception. The load-duration factor for impact (Table 11.5) does not apply to structural members pressure-treated with waterborne preservatives to the heavy retentions required for "marine" exposure or to structural members treated with fire-retardant chemicals.

To obtain preservative-treated glued-laminated timber, lumber may be treated before gluing and the members then glued to the desired size and shape. The already glued and machined members may be treated with certain treatments. When laminated members do not lend themselves to treatment because of size and shape, gluing of treated laminations is the only method of producing adequately treated members.

There are problems in gluing some treated woods. Certain combinations of adhesive, treatment, and wood species are compatible; other combinations are not. All adhesives of the same type do not produce bonds of equal quality for a particular wood species and preservative. The bonding of treated wood depends on the concentration of preservative on the surface at the time of gluing and the chemical effects of the preservative on the adhesive. In general, longer curing times or higher curing temperatures, and modifications in assembly times, are needed for treated than for untreated wood to obtain comparable adhesive bonds (see also Art. 11.7).

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 shown in Table 11.28 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.

Oil-borne chemicals are organic compounds dissolved in a suitable 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. There is a slight odor from such treatment, but it is usually not objectionable.

Waterborne inorganic salts are dissolved in water or aqua ammonia, which evaporates after treatment and leaves the chemicals in 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.

When treatment before gluing is required, waterborne salts, oil-borne chemicals in mineral spirits, or AWPA P9 volatile solvent are recommended. When treatment before gluing is not required or desired, creosote, creosote solutions, or oil-borne chemicals are recommended.

Table 11.28 Recommended Minimum Retentions of Preservatives, lb/ft*

Preservatives	Sawn and Laminated Timbers		Laminations		Sawn and Laminated Timbers		Laminations	
	Western Woods†	Southern Pine	Western Woods†	Southern Pine	Western Woods†	Southern Pine	Western Woods†	Southern Pine
Creosote or creosote solutions:								
Creosote	10	10	10	10	8	8	8	8
Creosote—coal-tar solution	10	10	NR‡	10	8	8	NR‡	8
Creosote-petroleum solution	12	NR‡	12	NR‡	6	NR‡	6	NR‡
Oil-borne chemicals:								
Pentachlorophenol (5% in specified petroleum oil)	0.6	0.6	0.6	0.6	0.3	0.3	0.3	0.3
Waterborne inorganic salts:								
Acid copper chromate (ACC)	NR‡	NR‡	0.50	0.50	0.25	0.25	0.25	0.25
Ammoniacal copper arsenite (ACA)	0.40	0.40	0.40	0.40	0.25	0.25	0.25	0.25
Chromated zinc chloride (CZC)	NR‡	NR‡	NR‡	NR‡	0.45	0.45	0.45	0.45
Chromated copper arsenate (CCA)	0.40	0.40	0.40	0.40	0.25	0.25	0.25	0.25
Ammoniacal copper zinc arsenate (ACZA)	0.40	0.40	0.40	0.40	0.25	0.25	0.25	0.25

* See latest edition of AITC 109, "Treating Standard for Structural Timber Framing," American Institute of Timber Construction or AWPA Standards C2 and C28, American Wood Preservers Association.

† Douglas fir, western hemlock, western larch.

‡ NR = not recommended.

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

Fire-retardant treatment with approved chemicals can make wood highly resistant to the spread of fire. 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.

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. The load-duration factor for impact (Table 11.5) should not be applied to structural members pressure-treated with fire-retardant chemicals.