Chapter 7

Fastenings

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Contents

The strength and stability of any structure depend heavily on the fastenings that hold its parts together. One prime advantage of wood as a structural material is the ease with which wood structural parts can be joined together with a wide variety of fastenings—nails, spikes, screws, bolts, lag screws, drift pins, staples, and metal connectors of various types. For utmost rigidity, strength, and service, each type of fastening requires joint designs adapted to the strength properties of wood along and across the grain and to dimensional changes that may occur with changes in moisture content.

Maximum lateral resistance and safe design load values for small-diameter (nails, spikes, and wood screws) and large-diameter dowel-type fasteners (bolts, lag screws, and drift pins) were based on an empirical method prior to 1991. Research conducted during the 1980s resulted in lateral resistance values that are currently based on a yield model theory. This theoretical method was adapted for the 1991 edition of the *National Design Specification for Wood Construction* (NDS). Because literature and design procedures exist that are related to both the empirical and theoretical methods, we refer to the empirical method as pre-1991 and the theoretical method as post-1991 throughout this chapter. Withdrawal resistance methods have not changed, so the pre- and post-1991 refer only to lateral resistance.

The information in this chapter represents primarily Forest Products Laboratory research results. A more comprehensive discussion of fastenings is given in the American Society of Civil Engineers Manuals and Reports on Engineering Practice No. 84, *Mechanical Connections in Wood Structures*. The research results of this chapter are often modified for structural safety, based on judgment or experience, and thus information presented in design documents may differ from information presented in this chapter. Additionally, research by others serves as a basis for some current design criteria. Allowable stress design criteria are presented in the *National Design Specification for Wood Construction* published by the American Forest and Paper Association; limit states design criteria are presented in the *Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction* published by the American Society of Civil Engineers.

Nails

Nails are the most common mechanical fastenings used in wood construction. There are many types, sizes, and forms of nails (Fig. 7–1). The load equations presented in this chapter apply for bright, smooth, common steel wire nails driven into wood when there is no visible splitting. For nails other than common wire nails, the loads can be adjusted by factors given later in the chapter.

Nails in use resist withdrawal loads, lateral loads, or a combination of the two. Both withdrawal and lateral resistance are affected by the wood, the nail, and the condition of use. In general, however, any variation in these factors has a more pronounced effect on withdrawal resistance than on lateral resistance. The serviceability of joints with nails laterally loaded does not depend greatly on withdrawal resistance unless large joint distortion is tolerable.

The diameters of various penny or gauge sizes of bright common nails are given in Table 7–1. The penny size designation should be used cautiously. International nail producers sometimes do not adhere to the dimensions of Table 7–1. Thus penny sizes, although still widely used, are obsolete. Specifying nail sizes by length and diameter dimensions is recommended. Bright box nails are generally of the same length but slightly smaller diameter (Table 7–2), while cement-coated nails such as coolers, sinkers, and coated box nails are slightly shorter (3.2 mm (1/8 in.)) and of smaller diameter than common nails of the same penny size. Helically and annularly threaded nails generally have smaller diameters than common nails for the same penny size (Table 7–3).

Withdrawal Resistance

The resistance of a nail shank to direct withdrawal from a piece of wood depends on the density of the wood, the diameter of the nail, and the depth of penetration. The surface condition of the nail at the time of driving also influences the initial withdrawal resistance.

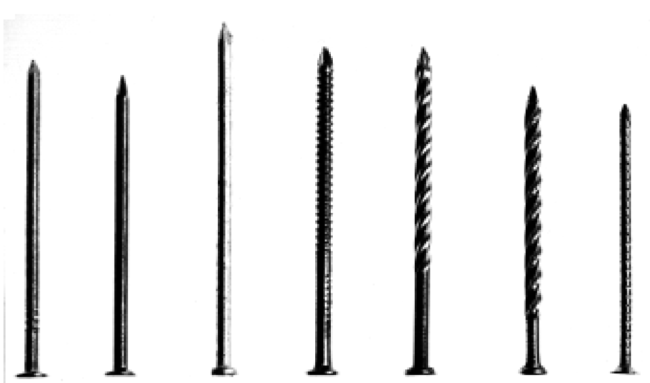

Figure 7–1. Various types of nails: (left to right) bright smooth wire nail, cement coated, zinc-coated, annularly threaded, helically threaded, helically threaded and barbed, and barbed.

Table 7–1. Sizes of bright common wire nails

Size	Gauge	Length (mm (in.))	Diameter (mm (in.))
6d	11-1/2	50.8 (2)	2.87 (0.113)
8d	10-1/4	63.5 (2-1/2)	3.33 (0.131)
10d	9	76.2 (3)	3.76 (0.148)
12d	9	82.6 (3-1/4)	3.76 (0.148)
16d	8	88.9 (3-1/2)	4.11 (0.162)
20d	6	101.6 (4)	4.88 (0.192)
30d	5	114.3 (4-1/2)	5.26 (0.207)
40d	4	127.0 (5)	5.72 (0.225)
50d	3	139.7 (5-1/2)	6.20 (0.244)
60d	2	152.4 (6)	6.65 (0.262)

Table 7–2. Sizes of smooth box nails

Size	Gauge	Length (mm (in.))	Diameter (mm (in.))
3d	14-1/2	31.8 (1-1/4)	1.93 (0.076)
4d	14	38.1 (1-1/2)	2.03 (0.080)
5d	14	44.5 (1-3/4)	2.03 (0.080)
6d	12-1/2	50.8 (2)	2.49 (0.098)
7d	12-1/2	57.2 (2-1/4)	2.49 (0.098)
8d	11-1/2	63.5 (2-1/2)	2.87 (0.113)
10d	10-1/2	76.2 (3)	3.25 (0.128)
16d	10	88.9 (3-1/2)	3.43 (0.135)
20d	9	101.6 (4)	3.76 (0.148)

Table 7–3. Sizes of helically and annularly threaded nails

Size	Length (mm (in.))	Diameter (mm (in.))
6d	50.8 (2)	3.05 (0.120)
8d	63.5 (2-1/2)	3.05 (0.120)
10d	76.2 (3)	3.43 (0.135)
12d	82.6 (3-1/4)	3.43 (0.135)
16d	88.9 (3-1/2)	3.76 (0.148)
20d	101.6 (4)	4.50 (0.177)
30d	114.3 (4-1/2)	4.50 (0.177)
40d	127.0 (5)	4.50 (0.177)
50d	139.7 (5-1/2)	4.50 (0.177)
60d	152.4 (6)	4.50 (0.177)
70d	177.8 (7)	5.26 (0.207)
80d	203.2 (8)	5.26 (0.207)
90d	228.6 (9)	5.26 (0.207)

For bright common wire nails driven into the side grain of seasoned wood or unseasoned wood that remains wet, the results of many tests have shown that the maximum withdrawal load is given by the empirical equation

$$p = 54.12G^{5/2}DL \qquad \text{(metric)} \qquad (7\text{–}1a)$$

$$p = 7,850G^{5/2}DL \qquad \text{(inch–pound)} \qquad (7\text{–}1b)$$

where p is maximum load (N, lb), L depth (mm, in.) of penetration of the nail in the member holding the nail point, G specific gravity of the wood based on ovendry weight and volume at 12% moisture content (see Ch. 4, Tables 4–2 to 4–5), and D diameter of the nail (mm, in.). (The NDS and LRFD use ovendry weight and volume as a basis.)

The loads expressed by Equation (7–1) represent average data. Certain wood species give test values that are somewhat greater or less than the equation values. A typical load–displacement curve for nail withdrawal (Fig. 7–2) shows that maximum load occurs at relatively small values of displacement.

Although the equation for nail-withdrawal resistance indicates that the dense, heavy woods offer greater resistance to nail withdrawal than do the lower density ones, lighter species should not be disqualified for uses requiring high resistance to withdrawal. As a rule, the less dense species do not split as readily as the denser ones, thus offering an opportunity for increasing the diameter, length, and number of the nails to compensate for the wood's lower resistance to nail withdrawal.

The withdrawal resistance of nail shanks is greatly affected by such factors as type of nail point, type of shank, time the nail remains in the wood, surface coatings, and moisture content changes in the wood.

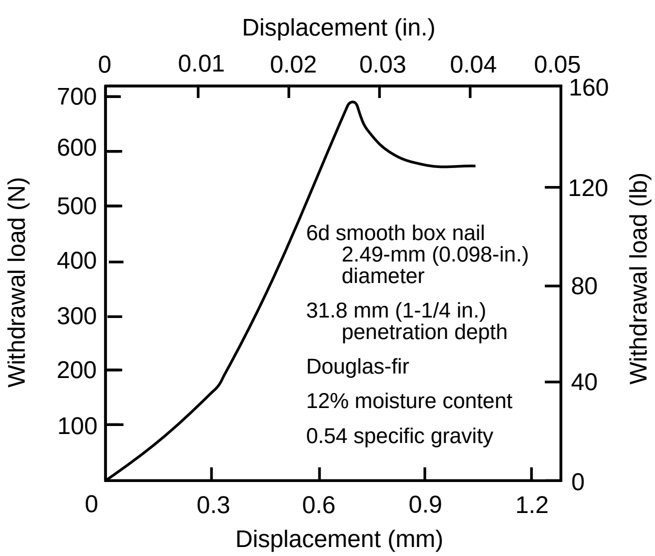

Figure 7–2. Typical load–displacement curve for direct withdrawal of a nail.

Effect of Seasoning

With practically all species, nails driven into green wood and pulled before any seasoning takes place offer about the same withdrawal resistance as nails driven into seasoned wood and pulled soon after driving. However, if common smooth-shank nails are driven into green wood that is allowed to season, or into seasoned wood that is subjected to cycles of wetting and drying before the nails are pulled, they lose a major part of their initial withdrawal resistance. The withdrawal resistance for nails driven into wood that is subjected to changes in moisture content may be as low as 25% of the values for nails tested soon after driving. On the other hand, if the wood fibers deteriorate or the nail corrodes under some conditions of moisture variation and time, withdrawal resistance is erratic; resistance may be regained or even increased over the immediate withdrawal resistance. However, such sustained performance should not be relied on in the design of a nailed joint.

In seasoned wood that is not subjected to appreciable moisture content changes, the withdrawal resistance of nails may also diminish due to relaxation of the wood fibers with time. Under all these conditions of use, the withdrawal resistance of nails differs among species and shows variation within individual species.

Effect of Nail Form

The surface condition of nails is frequently modified during the manufacturing process to improve withdrawal resistance. Such modification is usually done by surface coating, surface roughening, or mechanical deformation of the shank. Other factors that affect the surface condition of the nail are the oil film remaining on the shank after manufacture or corrosion resulting from storage under adverse conditions; but these factors are so variable that their influence on withdrawal resistance cannot be adequately evaluated.

Surface Modifications—A common surface treatment for nails is the so-called cement coating. Cement coatings, contrary to what the name implies, do not include cement as an ingredient; they generally are a composition of resin applied to the nail to increase the resistance to withdrawal by increasing the friction between the nail and the wood. If properly applied, they increase the resistance of nails to withdrawal immediately after the nails are driven into the softer woods. However, in the denser woods (such as hard maple, birch, or oak), cement-coated nails have practically no advantage over plain nails, because most of the coating is removed in driving. Some of the coating may also be removed in the side member before the nail penetrates the main member.

Good-quality cement coatings are uniform, not sticky to the touch, and cannot be rubbed off easily. Different techniques of applying the cement coating and variations in its ingredients may cause large differences in the relative resistance to withdrawal of different lots of cement-coated nails. Some nails may show only a slight initial advantage over plain nails. In the softer woods, the increase in withdrawal resistance of cement-coated nails is not permanent but drops off significantly after a month or so. Cement-coated nails are used primarily in construction of boxes, crates, and other containers usually built for rough handling and relatively short service.

Nails that have galvanized coatings, such as zinc, are intended primarily for uses where corrosion and staining resistance are important factors in permanence and appearance. If the zinc coating is evenly applied, withdrawal resistance may be increased, but extreme irregularities of the coating may actually reduce it. The advantage that uniformly coated galvanized nails may have over nongalvanized nails in resistance to initial withdrawal is usually reduced by repeated cycles of wetting and drying.

Nails have also been made with plastic coatings. The usefulness and characteristics of these coatings are influenced by the quality and type of coating, the effectiveness of the bond between the coating and base fastener, and the effectiveness of the bond between the coating and wood fibers. Some plastic coatings appear to resist corrosion or improve resistance to withdrawal, while others offer little improvement.

Fasteners with properly applied nylon coating tend to retain their initial resistance to withdrawal compared with other coatings, which exhibit a marked decrease in withdrawal resistance within the first month after driving.

A chemically etched nail has somewhat greater withdrawal resistance than some coated nails, as the minutely pitted surface is an integral part of the nail shank. Under impact loading, however, the withdrawal resistance of etched nails is little different from that of plain or cement-coated nails under various moisture conditions.

Sand-blasted nails perform in much the same manner as chemically etched nails.

Shape Modifications—Nail shanks may be varied from a smooth, circular form to give an increase in surface area without an increase in nail weight. Special nails with barbed, helically or annularly threaded, and other irregular shanks (Fig. 7–1) are commercially available.

The form and magnitude of the deformations along the shank influence the performance of the nails in various wood species. In wood remaining at a uniform moisture content, the withdrawal resistance of these nails is generally somewhat greater than that of common wire nails of the same diameter. For instance, annular-shank nails have about 40% greater resistance to withdrawal than common nails. However, under conditions involving changes in moisture content of the wood, some special nail forms provide considerably greater withdrawal resistance than the common wire nail—about four times greater for annularly and helically threaded nails of the same diameter. This is especially true of nails driven into green wood that subsequently dries. In general, annularly threaded nails sustain larger withdrawal loads, and helically threaded nails sustain greater impact withdrawal work values than do the other nail forms.

Nails with deformed shanks are sometimes hardened by heat treatments for use where driving conditions are difficult or to obtain improved performance, such as in pallet assembly. Hardened nails are brittle and care should be exercised to avoid injuries from fragments of nails broken during driving.

Nail Point—A smooth, round shank nail with a long, sharp point will usually have a greater withdrawal resistance, particularly in the softer woods, than the common wire nail (which usually has a diamond point). However, sharp points accentuate splitting in certain species, which may reduce withdrawal resistance. A blunt or flat point without taper reduces splitting, but its destruction of the wood fibers when driven reduces withdrawal resistance to less than that of the common wire nail. A nail tapered at the end and terminating in a blunt point will cause less splitting. In heavier woods, such a tapered, blunt-pointed nail will provide about the same withdrawal resistance, but in less dense woods, its resistance to withdrawal is less than that of the common nail.

Nail Head—Nail head classifications include flat, oval, countersunk, deep-countersunk, and brad. Nails with all types of heads, except the deep-countersunk, brad, and some of the thin flathead nails, are sufficiently strong to withstand the force required to pull them from most woods in direct withdrawal. The deep-countersunk and brad nails are usually driven below the wood surface and are not intended to carry large withdrawal loads. In general, the thickness and diameter of the heads of the common wire nails increase as the size of the nail increases.

The development of some pneumatically operated portable nailers has introduced nails with specially configured heads, such as T-nails and nails with a segment of the head cut off.

Corrosion and Staining

In the presence of moisture, metals used for nails may corrode when in contact with wood treated with certain preservative or fire-retardant salts (Chs. 14 and 17). Use of certain metals or metal alloys will reduce the amount of corrosion. Nails of copper, silicon bronze, and 304 and 316 stainless steel have performed well in wood treated with ammoniacal copper arsenate and chromated copper arsenate. The choice of metals for use with fire-retardant-treated woods depends upon the particular fire-retardant chemical.

Staining caused by the reaction of certain wood extractives (Ch. 3) and steel in the presence of moisture is a problem if appearance is important, such as with naturally finished siding. Use of stainless steel, aluminum, or hot-dipped galvanized nails can alleviate staining.

In general, the withdrawal resistance of copper and other alloy nails is comparable with that of common steel wire nails when pulled soon after driving.

Driving

The resistance of nails to withdrawal is generally greatest when they are driven perpendicular to the grain of the wood. When the nail is driven parallel to the wood fibers (that is, into the end of the piece) withdrawal resistance in the softer woods drops to 75% or even 50% of the resistance obtained when the nail is driven perpendicular to the grain. The difference between side- and end-grain withdrawal loads is less for dense woods than for softer woods. With most species, the ratio between the end- and side-grain withdrawal loads of nails pulled after a time interval, or after moisture content changes have occurred, is usually somewhat greater than that of nails pulled immediately after driving.

Toe nailing, a common method of joining wood framework, involves slant driving a nail or group of nails through the end or edge of an attached member and into a main member. Toe nailing requires greater skill in assembly than does ordinary end nailing but provides joints of greater strength and stability. Tests show that the maximum strength of toenailed joints under lateral and uplift loads is obtained by (a) using the largest nail that will not cause excessive splitting, (b) allowing an end distance (distance from the end of the attached member to the point of initial nail entry) of approximately one-third the length of the nail, (c) driving the nail at a slope of 30° with the attached member, and (d) burying the full shank of the nail but avoiding excessive mutilation of the wood from hammer blows.

The results of withdrawal tests with multiple nail joints in which the piece attached is pulled directly away from the main member show that slant driving is usually superior to straight driving when nails are driven into drywood and pulled immediately, and decidedly superior when nails are driven into green or partially dry wood that is allowed to season for a month or more. However, the loss in depth of penetration due to slant driving may, in some types of joints, offset the advantages of slant nailing. Cross slant driving of groups of nails through the side grain is usually somewhat more effective than parallel slant driving through the end grain.

Nails driven into lead holes with a diameter slightly smaller (approximately 90%) than the nail shank have somewhat greater withdrawal resistance than nails driven without lead holes. Lead holes also prevent or reduce splitting of the wood, particularly for dense species.

Clinching

The withdrawal resistance of smooth-shank, clinched nails is considerably greater than that of unclinched nails. The point of a clinched nail is bent over where the nail protrudes through the side member. The ratio between the loads for clinched and unclinched nails varies enormously, depending upon the moisture content of the wood when the nail is driven and withdrawn, the species of wood, the size of nail, and the direction of clinch with respect to the grain of the wood.

In dry or green wood, a clinched nail provides 45% to 170% more withdrawal resistance than an unclinched nail when withdrawn soon after driving. In green wood that seasons after a nail is driven, a clinched nail gives 250% to 460% greater withdrawal resistance than an unclinched nail. However, this improved strength of a clinched-nail joint does not justify the use of green lumber, because the joints may loosen as the lumber seasons. Furthermore, laboratory tests were made with single nails, and the effects of drying, such as warping, twisting, and splitting, may reduce the efficiency of a joint that has more than one nail. Clinching of nails is generally confined to such construction as boxes and crates and other container applications.

Nails clinched across the grain have approximately 20% more resistance to withdrawal than nails clinched along the grain.

Fastening of Plywood

The nailing characteristics of plywood are not greatly different from those of solid wood except for plywood’s greater resistance to splitting when nails are driven near an edge. The nail withdrawal resistance of plywood is 15% to 30% less than that of solid wood of the same thickness. The reason is that fiber distortion is less uniform in plywood than in solid wood. For plywood less than 12.5 mm (1/2-in.) thick, the greater splitting resistance tends to offset the lower withdrawal resistance compared with solid wood. The withdrawal resistance per unit length of penetration decreases as the number of plies per unit length increases. The direction of the grain of the face ply has little influence on the withdrawal resistance from the face near the end or edge of a piece of plywood. The direction of the grain of the face ply may influence the pull-through resistance of staples or nails with severely modified heads, such as T-heads. Fastener design information for plywood is available from APA–The Engineered Wood Association.

Allowable Loads

The preceding discussion dealt with maximum withdrawal loads obtained in short-time test conditions. For design, these loads must be reduced to account for variability, duration-of-load effects, and safety. A value of one-sixth the average maximum load has usually been accepted as the allowable load for long-time loading conditions. For normal duration of load, this value may be increased by 10%. Normal duration of load is defined as a load of 10-year duration.

Lateral Resistance

Pre-1991

Test loads at joint slips of 0.38 mm (0.015 in.) (approximate proportional limit load) for bright common wire nails in lateral resistance driven into the side grain (perpendicular to the wood fibers) of seasoned wood are expressed by the empirical equation

$$p = KD^{3/2} \tag{7–2}$$

where p is lateral load per nail, K a coefficient, and D diameter of the nail. Values of coefficient K are listed in Table 7–4 for ranges of specific gravity of hardwoods and softwoods. The loads given by the equation apply only where the side member and the member holding the nail point are of

Table 7–4. Coefficients for computing test loads for fasteners in seasoned wood[a] (pre-1991)

Specific gravity range[b]	Lateral load coefficient K (metric (inch–pound))					
	Nails[c]		Screws		Lag screws	
			Hardwoods			
0.33–0.47	50.04	(1,440)	23.17	(3,360)	26.34	(3,820)
0.48–0.56	69.50	(2,000)	31.99	(4,640)	29.51	(4,280)
0.57–0.74	94.52	(2,720)	44.13	(6,400)	34.13	(4,950)
			Softwoods			
0.29–0.42	50.04	(1,440)	23.17	(3,360)	23.30	(3,380)
0.43–0.47	62.55	(1,800)	29.79	(4,320)	26.34	(3,820)
0.48–0.52	76.45	(2,200)	36.40	(5,280)	29.51	(4,280)

[a]Wood with a moisture content of 15%.
[b]Specific gravity based on ovendry weight and volume at 12% moisture content.
[c]Coefficients based on load at joint slip of 0.38 mm (0.015 in.)

approximately the same density. The thickness of the side member should be about one-half the depth of penetration of the nail in the member holding the point.

The ultimate lateral nail loads for softwoods may approach 3.5 times the loads expressed by the equation, and for hardwoods they may be 7 times as great. The joint slip at maximum load, however, is more than 20 times 0.38 mm (0.015 in.). This is demonstrated by the typical load–slip curve shown in Figure 7–3. To maintain a sufficient ratio between ultimate load and the load at 0.38 mm (0.015 in.), the nail should penetrate into the member holding the point by not less than 10 times the nail diameter for dense woods (specific gravity greater than 0.61) and 14 times the diameter for low density woods (specific gravity less than 0.42). For species having densities between these two ranges, the penetration may be found by straight line interpolation.

Post-1991

The yield model theory selects the worst case of yield modes based on different possibilities of wood bearing and nail bending. It does not account for nail head effects. A description of the various combinations is given in Figure 7–4. Mode I is a wood bearing failure in either the main or side member; mode II is a rotation of the fastener in the joint without bending; modes III and IV are a combination of wood bearing failure and one or more plastic hinge yield formations in the fastener. Modes I_m and II have not been observed in nail and spike connections. The yield model theory is applicable to all types of dowel fasteners (nails, screws, bolts, lag screws) , and thus the wood bearing capacity is described by a material property called the dowel bearing strength.

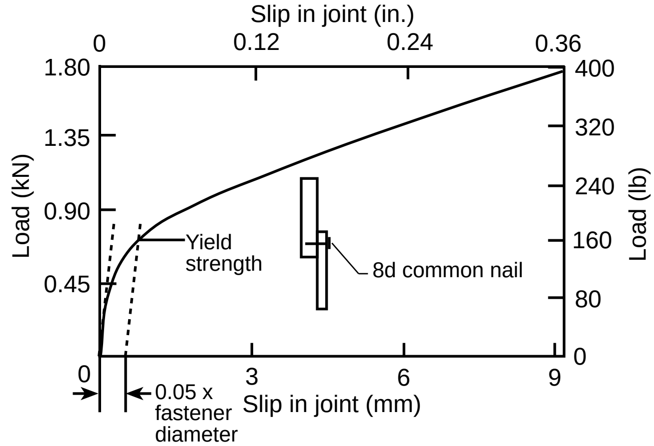

Figure 7–3. Typical relation between lateral load and slip in the joint and 5% offset definition.

The yield mode equations (Table 7–5) are entered with the dowel bearing strength and dimensions of the wood members and the bending yield strength and diameter of the fastener.

The dowel bearing strength of the wood is experimentally determined by compressing a dowel into a wood member. The strength basis is the load representing a 5% diameter offset on the load–deformation curve (Fig. 7–3). Dowel bearing strength F_e (Pa, lb/in^2) is empirically related to specific gravity G by

$$F_e = 114.5G^{1.84} \qquad \text{(metric)} \qquad (7\text{–}3a)$$

$$F_e = 16,600G^{1.84} \qquad \text{(inch–pound)} \qquad (7\text{–}3b)$$

where specific gravity is based on ovendry weight and volume.

Spacing

End distance, edge distance, and spacing of nails should be such as to prevent unusual splitting. As a general rule, nails should be driven no closer to the edge of the side member than one-half its thickness and no closer to the end than the thickness of the piece. Smaller nails can be driven closer to the edges or ends than larger ones because they are less likely to split the wood.

Grain Direction Effects

The lateral load for side-grain nailing applies whether the load is in a direction parallel to the grain of the pieces joined or at right angles to it. When nails are driven into the end grain (parallel with the wood fibers), limited data on softwood species indicate that their maximum resistance to lateral displacement is about two-thirds that for nails driven into the side grain. Although the average proportional limit loads appear to be about the same for end- and side-grain nailing, the individual results are more erratic for end-grain nailing, and the minimum loads approach only 75% of corresponding values for side-grain nailing.

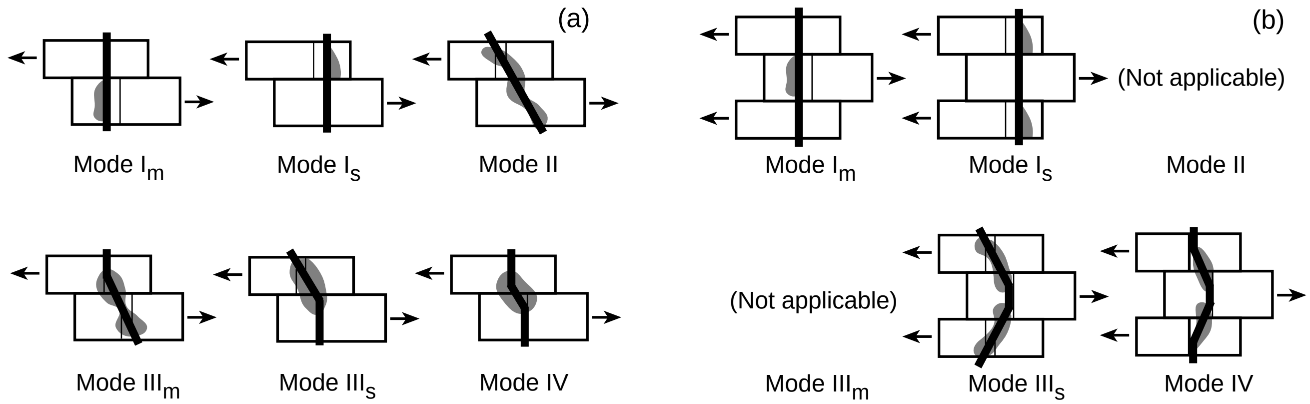

Figure 7–4. Various combinations of wood-bearing and fastener-bending yields for (a) two-member connections and (b) three-member connections.

Moisture Content Effects

Nails driven into the side grain of unseasoned wood give maximum lateral resistance loads approximately equal to those obtained in seasoned wood, but the lateral resistance loads at 0.38 mm (0.015 in.) joint slip are somewhat less. To prevent excessive deformation, lateral loads obtained for seasoned wood should be reduced by 25% for unseasoned wood that will remain wet or be loaded before seasoning takes place.

When nails are driven into green wood, their lateral proportional limit loads after the wood has seasoned are also less than when they are driven into seasoned wood and loaded. The erratic behavior of a nailed joint that has undergone one or more moisture content changes makes it difficult to establish a lateral load for a nailed joint under these conditions. Structural joints should be inspected at intervals, and if it is apparent that the joint has loosened during drying, the joint should be reinforced with additional nails.

Deformed-Shank Nails

Deformed-shank nails carry somewhat higher maximum lateral loads than do the same pennyweight common wire nails, but both perform similarly at small distortions in the joint. It should be noted that the same pennyweight deformed-shank nail has a different diameter than that of the common wire nail. These nails often have higher bending yield strength than common wire nails, resulting in higher lateral strength in modes III and IV.

Lateral Load–Slip Models

A considerable amount of work has been done to describe, by mathematical models, the lateral load–slip curve of nails. These models have become important because of their need as input parameters for advanced methods of structural analysis.

One theoretical model, which considers the nail to be a beam supported on an elastic foundation (the wood), describes the initial slope of the curve:

$$\delta = P\left[2(L_1 + L_2) - \frac{(J_1 - J_2)^2}{(K_1 + K_2)}\right] \qquad (7\text{–}4)$$

where P is the lateral load and δ is the joint slip. The factors L_1, L_2, J_1, J_2, K_1, and K_2 (Table 7–6) are combinations of hyperbolic and trigonometric functions of the quantities $\lambda_1 a$ and $\lambda_2 b$ in which a and b are the depth of penetration of the nail in members 1 and 2, respectively. For smooth round nails,

$$\lambda = 2\sqrt[4]{\frac{k_0}{\pi E D^3}} \qquad (7\text{–}5)$$

where k_0 is elastic bearing constant, D nail diameter, and E modulus of elasticity of the nail. For seasoned wood, the elastic bearing constant k_0 (N/mm^3, lb/in^3) has been shown to be related to average species specific gravity G if no lead hole is used by

$$k_0 = 582G \quad \text{(metric)} \qquad (7\text{–}6a)$$

$$k_0 = 2{,}144{,}000G \quad \text{(inch–pound} \qquad (7\text{–}6b)$$

If a prebored lead hole equal to 90% of the nail diameter is used,

$$k_0 = 869G \quad \text{(metric)} \qquad (7\text{–}7a)$$

$$k_0 = 3{,}200{,}000G \quad \text{(inch–pound)} \qquad (7\text{–}7b)$$

Other empirically derived models attempt to describe the entire load–slip curve. One such expression is

$$P = A \log_{10}(1 + B\delta) \qquad (7\text{–}8)$$

where the parameters A and B are empirically fitted.

Table 7–5. The 5% offset lateral yield strength (*Z*) for nails and screws for a two-member joint

Mode	*Z* value for nails	*Z* value for screws
$\mathrm{I_s}$	Dt_sF_{es}	Dt_sF_{es}
$\mathrm{III_m}$	$\dfrac{k_1DpF_{em}}{1+2R_e}$	—
$\mathrm{III_s}$	$\dfrac{k_2Dt_sF_{em}}{2+R_e}$	$\dfrac{k_3Dt_sF_{em}}{2+R_e}$
IV	$D^2\sqrt{\dfrac{2F_{em}F_{yb}}{3(1+R_e)}}$	$D^2\sqrt{\dfrac{1.75F_{em}F_{yb}}{3(1+R_e)}}$

Definitions

- D nail, spike, or screw diameter, mm (in.) (for annularly threaded nails, D is thread-root diameter; for screws, D is either the shank diameter or the root diameter if the threaded portion of the screw is in the shear plane)
- F_{em} dowel bearing stress of main member (member holding point), kPa (lb/in^2)
- F_{es} dowel bearing stress of side member, kPa (lb/in^2)
- F_{yb} bending yield stress of nail, spike, or screw, kPa (lb/in^2)
- p penetration of nail or spike in main member, mm (in.)
- t_s thickness of side member, mm (in.)
- Z offset lateral yield strength
- R_e $= F_{em}/F_{es}$

$$k_1 = -1 + \sqrt{2(1+R_e) + \frac{2F_{yb}(1+2R_e)D^2}{3\,F_{em}\,p^2}}$$

$$k_2 = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{2F_{yb}(2+R_e)D^2}{3\,F_{em}t_s^2}}$$

$$k_3 = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{F_{yb}(2+R_e)D^2}{2\,F_{em}t_s^2}}$$

Spikes

Common wire spikes are manufactured in the same manner as common wire nails. They have either a chisel point or a diamond point and are made in lengths of 76 to 305 mm (3 to 12 in.). For corresponding lengths in the range of 76 to 152 (3 to 6 in.), they have larger diameters (Table 7–7) than common wire nails, and beyond the 60d size they are usually designated by diameter.

Table 7–6. Expressions for factors in Equation (7–4)

Factor	Expression[a]
L_1	$\dfrac{\lambda_1}{k_1}\dfrac{\sinh\lambda_1 a\ \cosh\lambda_1 a - \sin\lambda_1 a\ \cos\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}$
L_2	$\dfrac{\lambda_2}{k_2}\dfrac{\sinh\lambda_2 b\ \cosh\lambda_2 b - \sin\lambda_2 b\ \cos\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}$
J_1	$\dfrac{\lambda_1^2}{k_1}\dfrac{\sinh^2\lambda_1 a + \sin^2\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}$
J_2	$\dfrac{\lambda_2^2}{k_2}\dfrac{\sinh^2\lambda_2 b + \sin^2\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}$
K_1	$\dfrac{\lambda_1^3}{k_1}\dfrac{\sinh\lambda_1 a\ \cosh\lambda_1 a + \sin\lambda_1 a\ \cos\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}$
K_2	$\dfrac{\lambda_2^3}{k_2}\dfrac{\sinh\lambda_2 b\ \cosh\lambda_2 b + \sin\lambda_2 b\ \cos\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}$

[a]$k_1 = k_{01}d$ and $k_2 = k_{02}d$, where k_1 and k_2 are the foundation moduli of members 1 and 2, respectively.

The withdrawal and lateral resistance equations and limitations given for common wire nails are also applicable to spikes, except that in calculating the withdrawal load for spikes, the depth of penetration is taken as the length of the spike in the member receiving the point, minus two-thirds the length of the point.

Staples

Different types of staples have been developed with various modifications in points, shank treatment and coatings, gauge, crown width, and length. These fasteners are available in clips or magazines for use in pneumatically operated portable staplers. Most factors that affect the withdrawal and lateral loads of nails similarly affect the loads on staples. The withdrawal resistance, for example, varies almost directly with the circumference and depth of penetration when the type of point and shank are similar to nails. Thus, Equation (7–1) has been used to predict the withdrawal load for one leg of a staple, but no verification tests have been done.

The load in lateral resistance varies approximately as the 3/2 power of the diameter when other factors, such as quality of metal, type of shank, and depth of penetration, are similar to nails. The diameter of each leg of a two-legged staple must therefore be about two-thirds the diameter of a nail to provide a comparable load. Equation (7–2) has been used to predict the lateral resistance of staples. However, yield model theory equations have not yet been experimentally verified for staples.

Table 7–7. Sizes of common wire spikes

Size	Length (mm (in.))	Diameter (mm (in.))
10d	76.2 (3)	4.88 (0.192)
12d	82.6 (3-1/4)	4.88 (0.192)
16d	88.9 (3-1/2)	5.26 (0.207)
20d	101.6 (4)	5.72 (0.225)
30d	114.3 (4-1/2)	6.20 (0.244)
40d	127.0 (5)	6.68 (0.263)
50d	139.7 (5-1/2)	7.19 (0.283)
60d	152.4 (6)	7.19 (0.283)
5/16 in.	177.8 (7)	7.92 (0.312)
3/8 in.	215.9 (8-1/2)	9.53 (0.375)

In addition to the immediate performance capability of staples and nails as determined by test, factors such as corrosion, sustained performance under service conditions, and durability in various uses should be considered in evaluating the relative usefulness of a stapled connection.

Drift Bolts

A drift bolt (or drift pin) is a long pin of iron or steel, with or without head or point. It is driven into a bored hole through one timber and into an adjacent one, to prevent the separation of the timbers connected and to transmit lateral load. The hole in the second member is drilled sufficiently deep to prevent the pin from hitting the bottom.

The ultimate withdrawal load of a round drift bolt or pin from the side grain of seasoned wood is given by

$$p = 45.51G^2DL \qquad \text{(metric)} \qquad (7\text{–}9a)$$

$$p = 6,600G^2DL \qquad \text{(inch–pound)} \qquad (7\text{–}9b)$$

where p is the ultimate withdrawal load (N, lb), G specific gravity based on the ovendry weight and volume at 12% moisture content of the wood, D diameter of the drift bolt (mm, in.), and L length of penetration of the bolt (mm, in.). (The NDS and LRFD use ovendry weight and volume as a basis.)

This equation provides an average relationship for all species, and the withdrawal load for some species may be above or below the equation values. It also presumes that the bolts are driven into prebored holes having a diameter 3.2 mm (1/8 in.) less than the bolt diameter.

Data are not available on lateral resistance of drift bolts. The yield model should provide lateral strength prediction, but the model has not been experimentally verified for drift bolts. Designers have used bolt data and design methods based on experience. This suggests that the load for a drift bolt driven into the side grain of wood should not exceed, and ordinarily should be taken as less than, that for a bolt of the same diameter. Bolt design values are based on the thickness of the main member in a joint. Thus the depth of penetration of the drift bolt must be greater than or equal to the main-member thickness on which the bolt design value is based. However, the drift bolt should not fully penetrate its joint.

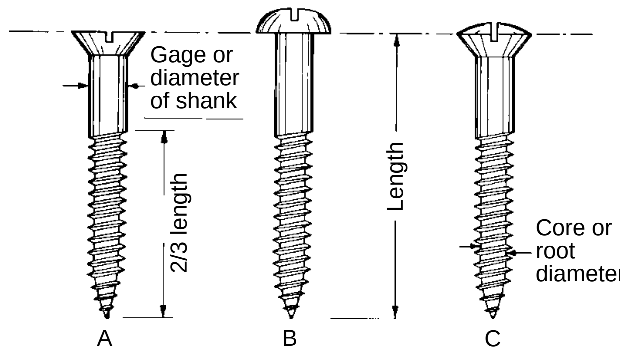

Figure 7–5. Common types of wood screws: A, flathead; B, roundhead; and C, ovalhead.

Wood Screws

The common types of wood screws have flat, oval, or round heads. The flathead screw is most commonly used if a flush surface is desired. Ovalhead and roundhead screws are used for appearance, and roundhead screws are used when countersinking is objectionable. The principal parts of a screw are the head, shank, thread, and core (Fig. 7–5). The root diameter for most sizes of screws averages about two-thirds the shank diameter. Wood screws are usually made of steel, brass, other metals, or alloys, and may have specific finishes such as nickel, blued, chromium, or cadmium. They are classified according to material, type, finish, shape of head, and diameter or gauge of the shank.

Current trends in fastenings for wood also include tapping screws. Tapping screws have threads the full length of the shank and may have some advantage for certain specific uses.

Withdrawal Resistance

Experimental Loads

The resistance of wood screw shanks to withdrawal from the side grain of seasoned wood varies directly with the square of the specific gravity of the wood. Within limits, the withdrawal load varies directly with the depth of penetration of the threaded portion and the diameter of the screw, provided the screw does not fail in tension. The screw will fail in tension when its strength is exceeded by the withdrawal strength from the wood. The limiting length to cause a tension failure decreases as the density of the wood increases since the withdrawal strength of the wood increases with density. The longer lengths of standard screws are therefore superfluous in dense hardwoods.

The withdrawal resistance of type A tapping screws, commonly called sheet metal screws, is in general about 10% greater than that for wood screws of comparable diameter and length of threaded portion. The ratio between the withdrawal resistance of tapping screws and wood screws varies from 1.16 in denser woods, such as oak, to 1.05 in lighter woods, such as redwood.

Ultimate test values for withdrawal loads of wood screws inserted into the side grain of seasoned wood may be expressed as

$$p = 108.25G^2DL \quad \text{(metric)} \qquad (7\text{–}10a)$$

$$p = 15{,}700G^2DL \quad \text{(inch–pound)} \qquad (7\text{–}10b)$$

where p is maximum withdrawal load (N, lb), G specific gravity based on ovendry weight and volume at 12% moisture content, D shank diameter of the screw (mm, in.), and L length of penetration of the threaded part of the screw (mm, in.). (The NDS and LRFD use ovendry weight and volume as a basis.) These values are based on reaching ultimate load in 5- to 10-min.

This equation is applicable when screw lead holes have a diameter of about 70% of the root diameter of the threads in softwoods, and about 90% in hardwoods.

The equation values are applicable to the screw sizes listed in Table 7–8. (Shank diameters are related to screw gauges.)

For lengths and gauges outside these limits, the actual values are likely to be less than the equation values.

The withdrawal loads of screws inserted in the end grain of wood are somewhat erratic, but when splitting is avoided, they should average 75% of the load sustained by screws inserted in the side grain.

Lubricating the surface of a screw with soap or similar lubricant is recommended to facilitate insertion, especially in dense woods, and it will have little effect on ultimate withdrawal resistance.

Table 7–8. Screw sizes appropriate for Equation (7–10)

Screw length (mm (in.))	Gauge limits
12.7 (1/2)	1 to 6
19.0 (3/4)	2 to 11
25.4 (1)	3 to 12
38.1 (1-1/2)	5 to 14
50.8 (2)	7 to 16
63.5 (2-1/2)	9 to 18
76.2 (3)	12 to 20

Fastening of Particleboard

Tapping screws are commonly used in particleboard where withdrawal strength is important. Care must be taken when tightening screws in particleboard to avoid stripping the threads. The maximum amount of torque that can be applied to a screw before the threads in the particleboard are stripped is given by

$$T = 3.16 + 0.0096X \quad \text{(metric)} \qquad (7\text{–}11a)$$

$$T = 27.98 + 1.36X \quad \text{(inch–pound)} \qquad (7\text{–}11b)$$

where T is torque (N–m, in–lb) and X is density of the particleboard (kg/m^3, lb/ft^3). Equation (7–11) is for 8-gauge screws with a depth of penetration of 15.9 mm (5/8 in.). The maximum torque is fairly constant for lead holes of 0 to 90% of the root diameter of the screw.

Ultimate withdrawal loads P (N, lb) of screws from particleboard can be predicted by

$$P = KD^{1/2}(L - D/3)^{5/4}G^2 \qquad (7\text{–}12)$$

where D is shank diameter of the screw (mm, in.), L depth of embedment of the threaded portion of the screw (mm, in.), and G specific gravity of the board based on ovendry weight and volume at current moisture content. For metric measurements, $K = 41.1$ for withdrawal from the face of the board and $K = 31.8$ for withdrawal from the edge; for inch–pound measurements, $K = 2{,}655$ for withdrawal from the face and $K = 2{,}055$ for withdrawal from the edge. Equation (7–12) applies when the setting torque is between 60% to 90% of T (Eq. (7–11)).

Withdrawal resistance of screws from particleboard is not significantly different for lead holes of 50% to 90% of the root diameter. A higher setting torque will produce a somewhat higher withdrawal load, but there is only a slight difference (3%) in values between 60% to 90% setting torques (Eq. (7–11)). A modest tightening of screws in many cases provides an effective compromise between optimizing withdrawal resistance and stripping threads.

Equation (7–12) can also predict the withdrawal of screws from fiberboard with $K = 57.3$ (metric) or 3,700 (inch–pound) for the face and $K = 44.3$ (metric) or 2,860 (inch–pound) for the edge of the board.

Lateral Resistance

Pre-1991

The proportional limit loads obtained in tests of lateral resistance for wood screws in the side grain of seasoned wood are given by the empirical equation

$$p = KD^2 \qquad (7\text{–}13)$$

where p is lateral load, D diameter of the screw shank, and K a coefficient depending on the inherent characteristics of the wood species. Values of screw shank diameters for various screw gauges are listed in Table 7–9.

Table 7–9. Screw shank diameters for various screw gauges

Screw number or gauge	Diameter (mm (in.))
4	2.84 (0.112)
5	3.18 (0.125)
6	3.51 (0.138)
7	3.84 (0.151)
8	4.17 (0.164)
9	4.50 (0.177)
10	4.83 (0.190)
11	5.16 (0.203)
12	5.49 (0.216)
14	6.15 (0.242)
16	6.81 (0.268)
18	7.47 (0.294)
20	8.13 (0.320)
24	9.45 (0.372)

Values of K are based on ranges of specific gravity of hardwoods and softwoods and are given in Table 7–4. They apply to wood at about 15% moisture content. Loads computed by substituting these constants in the equation are expected to have a slip of 0.18 to 0.25 mm (0.007 to 0.010 in.), depending somewhat on the species and density of the wood.

Equation (7–13) applies when the depth of penetration of the screw into the block receiving the point is not less than seven times the shank diameter and when the side member and the main member are approximately of the same density. The thickness of the side member should be about one-half the depth of penetration of the screw in the member holding the point. The end distance should be no less than the side member thickness, and the edge distances no less than one-half the side member thickness.

This depth of penetration (seven times shank diameter) gives an ultimate load of about four times the load obtained by the equation. For a depth of penetration of less than seven times the shank diameter, the ultimate load is reduced about in proportion to the reduction in penetration, and the load at the proportional limit is reduced somewhat less rapidly. When the depth of penetration of the screw in the holding block is four times the shank diameter, the maximum load will be less than three times the load expressed by the equation, and the proportional limit load will be approximately equal to that given by the equation. When the screw holds metal to wood, the load can be increased by about 25%.

For these lateral loads, the part of the lead hole receiving the shank should be the same diameter as the shank or slightly smaller; that part receiving the threaded portion should be the same diameter as the root of the thread in dense species or slightly smaller than the root in low-density species.

Screws should always be turned in. They should never be started or driven with a hammer because this practice tears the wood fibers and injures the screw threads, seriously reducing the load carrying capacity of the screw.

Post-1991

Screw lateral strength is determined by the yield model theory (Table 7–5). Modes I, III, and IV failures may occur (Fig. 7–4). The dowel bearing strength values are based on the same specific gravity equation used to establish values for nails (Eq. (7–3)). Further discussion of screw lateral strength is found in ASCE Manual No. 84, *Mechanical Connections in Wood Structures.*

Lag Screws

Lag screws are commonly used because of their convenience, particularly where it would be difficult to fasten a bolt or where a nut on the surface would be objectionable. Commonly available lag screws range from about 5.1 to 25.4 mm (0.2 to 1 in.) in diameter and from 25.4 to 406 mm (1 to 16 in.) in length. The length of the threaded part varies with the length of the screw and ranges from 19.0 mm (3/4 in.) with the 25.4- and 31.8-mm (1- and 1-1/4-in.) screws to half the length for all lengths greater than 254 mm (10 in.). Lag screws have a hexagonal-shaped head and are tightened by a wrench (as opposed to wood screws, which have a slotted head and are tightened by a screw driver). The following equations for withdrawal and lateral loads are based on lag screws having a base metal average tensile yield strength of about 310.3 MPa (45,000 lb/in^2) and an average ultimate tensile strength of 530.9 MPa (77,000 lb/in^2).

Withdrawal Resistance

The results of withdrawal tests have shown that the maximum direct withdrawal load of lag screws from the side grain of seasoned wood may be computed as

$$p = 125.4G^{3/2}D^{3/4}L \qquad \text{(metric)} \qquad (7\text{–}14a)$$

$$p = 8{,}100G^{3/2}D^{3/4}L \qquad \text{(inch–pound)} \qquad (7\text{–}14b)$$

where p is maximum withdrawal load (N, lb), D shank diameter (mm, in.), G specific gravity of the wood based on ovendry weight and volume at 12% moisture content, and L length (mm, in.) of penetration of the threaded part. (The NDS and LRFD use ovendry weight and volume as a basis.) Equation (7–14) was developed independently of Equation (7–10) but gives approximately the same results.

Lag screws, like wood screws, require prebored holes of the proper size (Fig. 7–6). The lead hole for the shank should be the same diameter as the shank. The diameter of the lead hole for the threaded part varies with the density of the wood: For low-density softwoods, such as the cedars and white pines, 40% to 70% of the shank diameter; for Douglas-fir and Southern Pine, 60% to 75%; and for dense hardwoods, such as oaks, 65% to 85%. The smaller percentage in each range applies to lag screws of the smaller diameters and the larger

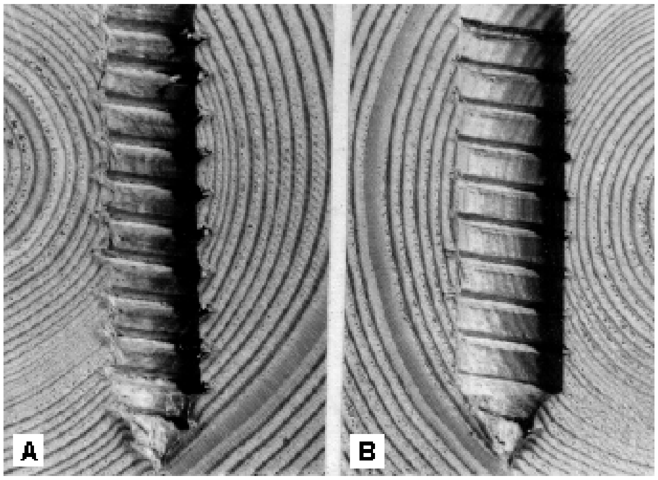

Figure 7–6. A, Clean-cut, deep penetration of thread made by lag screw turned into a lead hole of proper size, and B, rough, shallow penetration of thread made by lag screw turned into oversized lead hole.

percentage to lag screws of larger diameters. Soap or similar lubricants should be used on the screw to facilitate turning, and lead holes slightly larger than those recommended for maximum efficiency should be used with long screws.

In determining the withdrawal resistance, the allowable tensile strength of the lag screw at the net (root) section should not be exceeded. Penetration of the threaded part to a distance about seven times the shank diameter in the denser species (specific gravity greater than 0.61) and 10 to 12 times the shank diameter in the less dense species (specific gravity less than 0.42) will develop approximately the ultimate tensile strength of the lag screw. Penetrations at intermediate densities may be found by straight-line interpolation.

The resistance to withdrawal of a lag screw from the end-grain surface of a piece of wood is about three-fourths as great as its resistance to withdrawal from the side-grain surface of the same piece.

Lateral Resistance

Pre-1991

The experimentally determined lateral loads for lag screws inserted in the side grain and loaded parallel to the grain of a piece of seasoned wood can be computed as

$$p = KD^2 \tag{7–15}$$

where p is proportional limit lateral load (N, lb) parallel to the grain, K a coefficient depending on the species specific gravity, and D shank diameter of the lag screw (mm, in.). Values of K for a number of specific gravity ranges can be found in Table 7–4. These coefficients are based on average results for several ranges of specific gravity for hardwoods and softwoods. The loads given by this equation apply when the thickness of the side member is 3.5 times the shank diameter of the lag screw, and the depth of penetration in the main member is seven times the diameter in the harder woods and 11 times the diameter in the softer woods. For other thicknesses, the computed loads should be multiplied by the factors listed in Table 7–10.

The thickness of a solid wood side member should be about one-half the depth of penetration in the main member.

When the lag screw is inserted in the side grain of wood and the load is applied perpendicular to the grain, the load given by the lateral resistance equation should be multiplied by the factors listed in Table 7–11.

For other angles of loading, the loads may be computed from the parallel and perpendicular values by the use of the Scholten nomograph for determining the bearing strength of wood at various angles to the grain (Fig. 7–7).

Table 7–10. Multiplication factors for loads computed from Equation (7–15)

Ratio of thickness of side member to shank diameter of lag screw	Factor
2	0.62
2.5	0.77
3	0.93
3.5	1.00
4	1.07
4.5	1.13
5	1.18
5.5	1.21
6	1.22
6.5	1.22

Table 7–11. Multiplication factors for loads applied perpendicular to grain computed from Equation (7–15) with lag screw in side grain of wood

Shank diameter of lag screw (mm (in.))	Factor
4.8 (3/16)	1.00
6.4 (1/4)	0.97
7.9 (5/16)	0.85
9.5 (3/8)	0.76
11.1 (7/16)	0.70
12.7 (1/2)	0.65
15.9 (5/8)	0.60
19.0 (3/4)	0.55
22.2 (7/8)	0.52
25.4 (1)	0.50

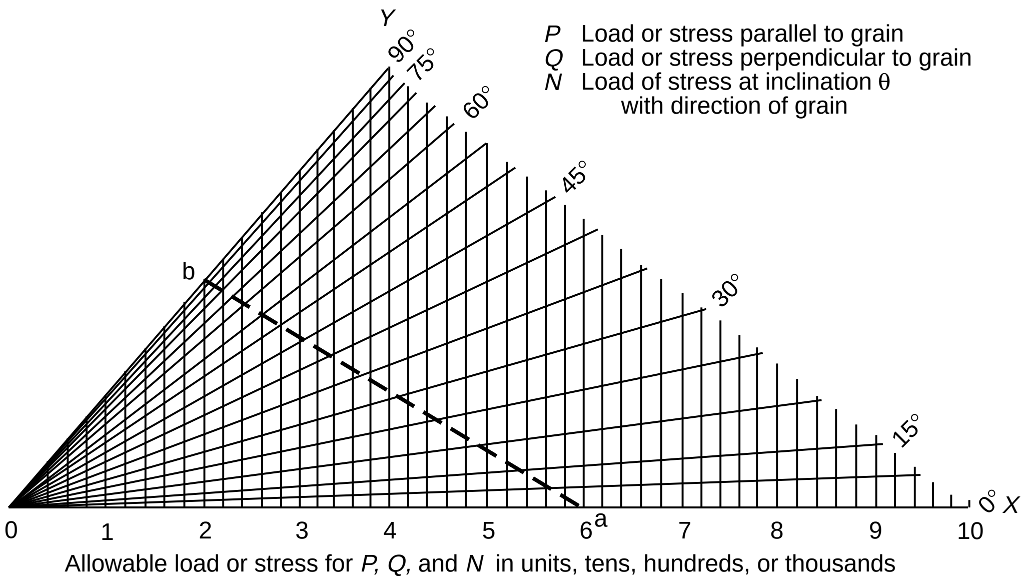

Figure 7–7. Scholten nomograph for determining the bearing stress of wood at various angles to the grain. The dashed line *ab* refers to the example given in the text.

The nomograph provides values as given by the Hankinson equation,

$$N = \frac{PQ}{P\sin^2\theta + Q\cos^2\theta} \qquad (7–16)$$

where P is load or stress parallel to the grain, Q load or stress perpendicular to the grain, and N load or stress at an inclination θ with the direction of the grain.

***Example**:* P, the load parallel to grain, is 6,000 lb, and Q, the load perpendicular to the grain, is 2,000 lb. The load at an angle of 40° to grain, N, is found as follows: Connect with a straight line 6,000 lb (a) on line OX of the nomograph with the intersection (b) on line OY of a vertical line through 2,000 lb. The point where line ab intersects the line representing the given angle 40° is directly above the load, 3,300 lb.

Values for lateral resistance as computed by the preceding methods are based on complete penetration of the unthreaded shank into the side member but not into the main member. When the shank penetrates the main member, the permitted increases in loads are given in Table 7–12.

When lag screws are used with metal plates, the lateral loads parallel to the grain may be increased 25%, provided the plate thickness is sufficient so that the bearing capacity of the steel is not exceeded. No increase should be made when the applied load is perpendicular to the grain.

Lag screws should not be used in end grain, because splitting may develop under lateral load. If lag screws are so used, however, the loads should be taken as two-thirds those for lateral resistance when lag screws are inserted into side grain and the loads act perpendicular to the grain.

The spacings, end and edge distances, and net section for lag screw joints should be the same as those for joints with bolts (discussed later) of a diameter equal to the shank diameter of the lag screw.

Lag screws should always be inserted by turning with a wrench, not by driving with a hammer. Soap, beeswax, or other lubricants applied to the screw, particularly with the denser wood species, will facilitate insertion and prevent damage to the threads but will not affect performance of the lag screw.

Post-1991

Lag screw lateral strength is determined by the yield model theory table similar to the procedure for bolts. Modes I, III, and IV yield may occur (Fig. 7–4). The dowel bearing

Table 7–12. Permitted increases in loads when lag screw unthreaded shank penetrates foundation member

Ratio of penetration of shank into foundation member to shank diameter	Increase in load (%)
1	8
2	17
3	26
4	33
5	36
6	38
7	39

strength values are based on the same parallel- and perpendicular-to-grain specific gravity equations used to establish values for bolts.

Bolts

Bearing Stress of Wood Under Bolts

The bearing stress under a bolt is computed by dividing the load on a bolt by the product *LD*, where *L* is the length of a bolt in the main member and *D* is the bolt diameter. Basic parallel-to-grain and perpendicular-to-grain bearing stresses have been obtained from tests of three-member wood joints where each side member is half the thickness of the main member. The side members were loaded parallel to grain for both parallel- and perpendicular-to-grain tests. Prior to 1991, bearing stress was based on test results at the proportional limit; since 1991, bearing stress is based on test results at a yield limit state, which is defined as the 5% diameter offset on the load–deformation curve (similar to Fig. 7–3).

The bearing stress at proportional limit load is largest when the bolt does not bend, that is, for joints with small *L/D* values. The curves of Figures 7–8 and 7–9 show the reduction in proportional limit bolt-bearing stress as *L/D* increases. The bearing stress at maximum load does not decrease as *L/D* increases, but remains fairly constant, which means that the ratio of maximum load to proportional limit load increases as *L/D i*ncreases. To maintain a fairly constant ratio between maximum load and design load for bolts, the relations between bearing stress and *L/D* ratio have been adjusted as indicated in Figures 7–8 and 7–9.

The proportional limit bolt-bearing stress parallel to grain for small *L/D* ratios is approximately 50% of the small clear crushing strength for softwoods and approximately 60% for hardwoods. For bearing stress perpendicular to the grain, the ratio between bearing stress at proportional limit load and the small clear proportional limit stress in compression perpendicular to grain depends upon bolt diameter (Fig. 7–10) for small *L/D* ratios.

Species compressive strength also affects the *L/D* ratio relationship, as indicated in Figure 7–9. Relatively higher bolt proportional-limit stress perpendicular to grain is obtained with wood low in strength (proportional limit stress of 3,930 kPa (570 lb/in^2) than with material of high strength (proportional limit stress of 7,860 kPa (1,140 lb/in^2)). This effect also occurs for bolt-bearing stress parallel to grain, but not to the same extent as for perpendicular-to-grain loading.

The proportional limit bolt load for a three-member joint with side members half the thickness of the main member may be estimated by the following procedures.

For parallel-to-grain loading, (a) multiply the species small clear compressive parallel strength (Tables 4–3, 4–4, or 4–5) by 0.50 for softwoods or 0.60 for hardwoods, (b) multiply this product by the appropriate factor from Figure 7–8 for the *L/D* ratio of the bolt, and (c) multiply this product by *LD*.

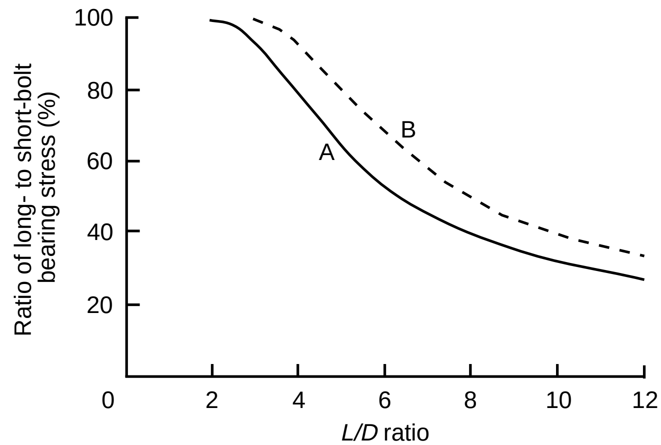

Figure 7–8. Variation in bolt-bearing stress at the proportional limit parallel to grain with *L/D* ratio. Curve A, relation obtained from experimental evaluation; curve B, modified relation used for establishing design loads.

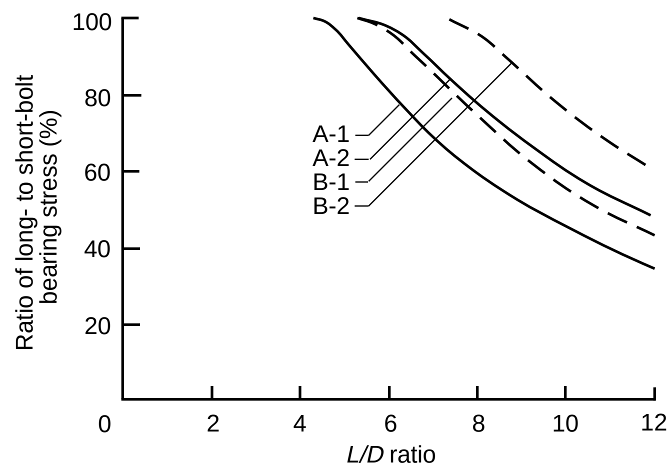

Figure 7–9. Variation in bolt-bearing stress at the proportional limit perpendicular to grain with *L/D* ratio. Relations obtained from experimental evaluation for materials with average compression perpendicular stress of 7,860 kPa (1,140 lb/in^2) (curve A–1) and 3,930 kPa (570 lb/in^2) (curve A–2). Curves B–1 and B–2, modified relations used for establishing design loads.

For perpendicular-to-grain loading, (a) multiply the species compression perpendicular-to-grain proportional limit stress (Tables 4–3, 4–4, or 4–5) by the appropriate factor from Figure 7–10, (b) multiply this product by the appropriate factor from Figure 7–9, and (c) multiply this product by *LD*.

Loads at an Angle to the Grain

For loads applied at an angle intermediate between those parallel to the grain and perpendicular to the grain, the bolt-bearing stress may be obtained from the nomograph in Figure 7–7.

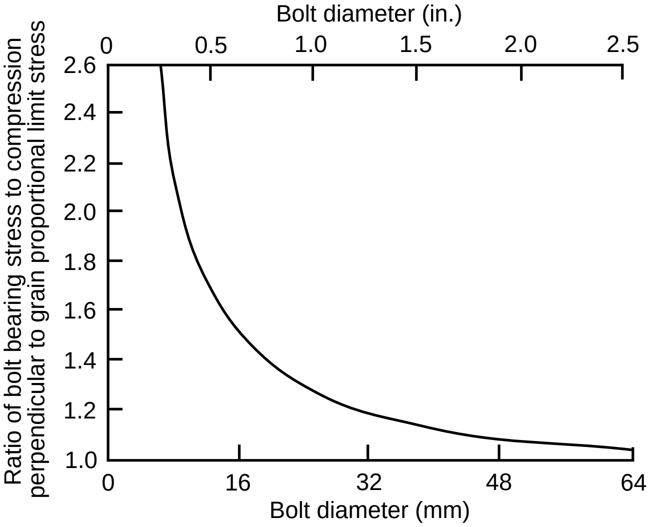

Figure 7–10. Bearing stress perpendicular to the grain as affected by bolt diameter.

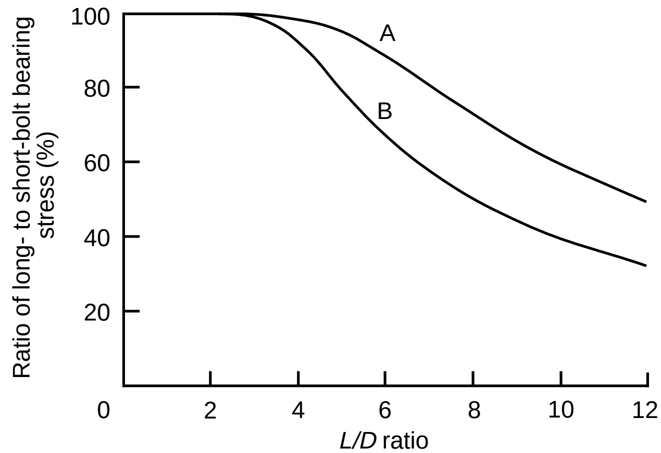

Figure 7–11. Variation in the proportional limit bolt-bearing stress parallel to grain with *L/D* ratio. Curve A, bolts with yield stress of 861.84 MPa (125,000 lb/in^2); curve B, bolts with yield stress of 310.26 MPa (45,000 lb/in^2).

Steel Side Plates

When steel side plates are used, the bolt-bearing stress parallel to grain at joint proportional limit is approximately 25% greater than that for wood side plates. The joint deformation at proportional limit is much smaller with steel side plates. If loads at equivalent joint deformation are compared, the load for joints with steel side plates is approximately 75% greater than that for wood side plates. Pre-1991 design criteria included increases in connection strength with steel side plates; post-1991 design criteria include steel side plate behavior in the yield model equations.

For perpendicular-to-grain loading, the same loads are obtained for wood and steel side plates.

Bolt Quality

Both the properties of the wood and the quality of the bolt are factors in determining the strength of a bolted joint. The percentages given in Figures 7–8 and 7–9 for calculating bearing stress apply to steel machine bolts with a yield stress of 310 MPa (45,000 lb/in^2). Figure 7–11 indicates the increase in bearing stress parallel to grain for bolts with a yield stress of 862 MPa (125,00 lb/in^2).

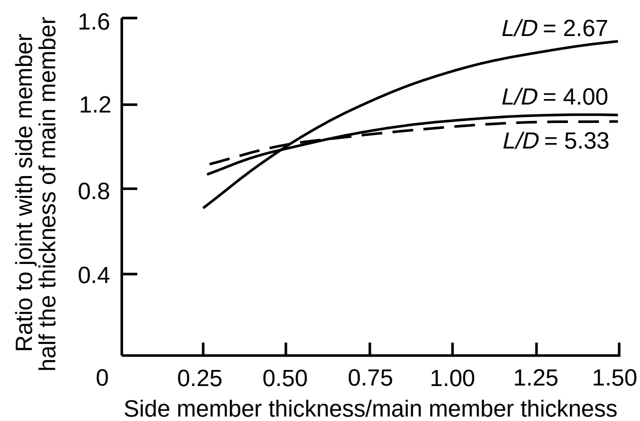

Figure 7–12. Proportional limit load related to side member thickness for three-member joints. Center member thickness was 50.8 mm (2 in.).

Effect of Member Thickness

The proportional limit load is affected by the ratio of the side member thickness to the main member thickness (Fig. 7–12).

Pre-1991 design values for bolts are based on joints with the side member half the thickness of the main member. The usual practice in design of bolted joints is to take no increase in design load when the side members are greater than half the thickness of the main member. When the side members are less than half the thickness of the main member, a design load for a main member that is twice the thickness of the side member is used. Post-1991 design values include member thickness directly in the yield model equations.

Two-Member, Multiple-Member Joints

In pre-1991 design, the proportional limit load was taken as half the load for a three-member joint with a main member the same thickness as the thinnest member for two-member joints.

For four or more members in a joint, the proportional limit load was taken as the sum of the loads for the individual shear planes by treating each shear plane as an equivalent two-member joint.

Post-1991 design for joints with four or more members also results in values per shear plane. Connection strength for any number of members is conservatively found by multiplying the value for the weakest shear plane by the number of shear planes.

Spacing, Edge, and End Distance

The center-to-center distance along the grain should be at least four times the bolt diameter for parallel-to-grain loading. The minimum center-to-center spacing of bolts in the across-the-grain direction for loads acting through metal side plates and parallel to the grain need only be sufficient to permit the tightening of the nuts. For wood side plates, the spacing is controlled by the rules applying to loads acting parallel to grain if the design load approaches the bolt-bearing capacity of the side plates. When the design load is less than the bolt-bearing capacity of the side plates, the spacing may be reduced below that required to develop their maximum capacity.

When a joint is in tension, the bolt nearest the end of a timber should be at a distance from the end of at least seven times the bolt diameter for softwoods and five times for hardwoods. When the joint is in compression, the end margin may be four times the bolt diameter for both softwoods and hardwoods. Any decrease in these spacings and margins will decrease the load in about the same ratio.

For bolts bearing parallel to the grain, the distance from the edge of a timber to the center of a bolt should be at least 1.5 times the bolt diameter. This margin, however, will usually be controlled by (a) the common practice of having an edge margin equal to one-half the distance between bolt rows and (b) the area requirements at the critical section. (The critical section is that section of the member taken at right angles to the direction of load, which gives the maximum stress in the member based on the net area remaining after reductions are made for bolt holes at that section.) For parallel-to-grain loading in softwoods, the net area remaining at the critical section should be at least 80% of the total area in bearing under all the bolts in the particular joint under consideration; in hardwoods it should be 100%.

For bolts bearing perpendicular to the grain, the margin between the edge toward which the bolt pressure is acting and the center of the bolt or bolts nearest this edge should be at least four times the bolt diameter. The margin at the opposite edge is relatively unimportant.

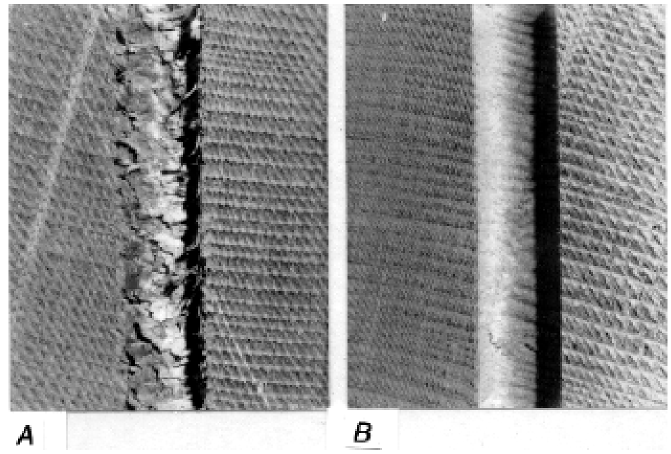

Figure 7–13. Effect of rate of feed and drill speed on the surface condition of bolt holes drilled in Sitka spruce. A, hole was bored with a twist drill rotating at a peripheral speed of 7.62 m/min (300 in/min); feed rate was 1.52 m/min (60 in/min). B, hole was bored with the same drill at a peripheral speed of 31.75 m/min (1,250 in/min); feed rate was 50.8 mm/min (2 in/min).

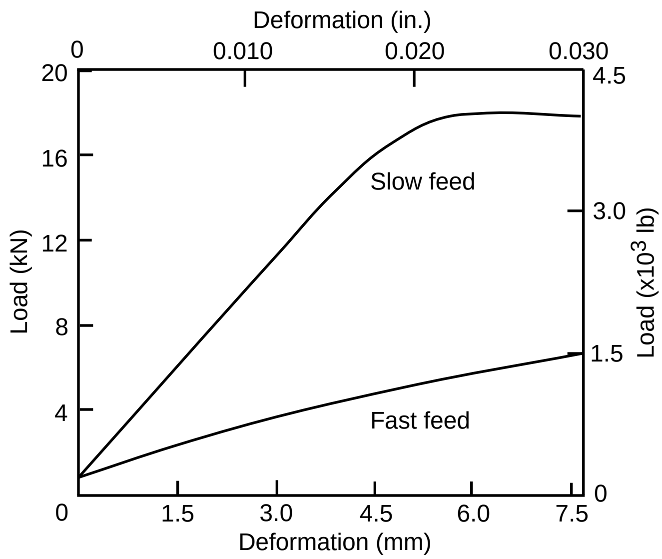

Figure 7–14. Typical load–deformation curves showing the effect of surface condition of bolt holes, resulting from a slow feed rate and a fast feed rate, on the deformation in a joint when subjected to loading under bolts. The surface conditions of the bolt holes were similar to those illustrated in Figure 7–13.

Effect of Bolt Holes

The bearing strength of wood under bolts is affected considerably by the size and type of bolt holes into which the bolts are inserted. A bolt hole that is too large causes nonuniform bearing of the bolt; if the bolt hole is too small, the wood will split when the bolt is driven. Normally, bolts should fit so that they can be inserted by tapping lightly with a wood mallet. In general, the smoother the hole, the higher the bearing values will be (Fig. 7–13). Deformations accompanying the load are also less with a smoother bolt-hole surface (Fig. 7–14).

Rough holes are caused by using dull bits and improper rates of feed and drill speed. A twist drill operated at a peripheral speed of approximately 38 m/min (1,500 in/min) produces uniformly smooth holes at moderate feed rates. The rate of feed depends upon the diameter of the drill and the speed of rotation but should enable the drill to cut rather than tear the wood. The drill should produce shavings, not chips.

Proportional limit loads for joints with bolt holes the same diameter as the bolt will be slightly higher than for joints with a 1.6-mm (1/16-in.) oversized hole. However, if drying takes place after assembly of the joint, the proportional limit load for snug-fitting bolts will be considerably less due to the effects of shrinkage.

Pre-1991 Allowable Loads

The following procedures are used to calculate allowable bolt loads for joints with wood side members, each half the thickness of the main member.

Parallel to Grain—The starting point for parallel-to-grain bolt values is the maximum green crushing strength for the species or group of species. Procedures outlined in ASTM D2555 are used to establish a 5% exclusion value. The exclusion value is divided by a factor of 1.9 to adjust to a 10-year normal duration of load and provide a factor of safety. This value is multiplied by 1.20 to adjust to a seasoned strength. The resulting value is called the basic bolt-bearing stress parallel to grain.

The basic bolt-bearing stress is then adjusted for the effects of *L/D* ratio. Table 7–13 gives the percentage of basic stress for three classes of species. The particular class for the species is determined from the basic bolt-bearing stress as indicated in Table 7–14. The adjusted bearing stress is further multiplied

Table 7–13. Percentage of basic bolt-bearing stress used for calculating allowable bolt loads

Ratio of bolt length to diameter (*L/D*)	*L/D* adjustment factor by class[a]						
	Parallel to grain			Perpendicular to grain			
	1	2	3	1	2	3	4
1	100.0	100.0	100.0	100.0	100.0	100.0	100.0
2	100.0	100.0	100.0	100.0	100.0	100.0	100.0
3	100.0	100.0	99.0	100.0	100.0	100.0	100.0
4	99.5	97.4	92.5	100.0	100.0	100.0	100.0
5	95.4	88.3	80.0	100.0	100.0	100.0	100.0
6	85.6	75.8	67.2	100.0	100.0	100.0	96.3
7	73.4	65.0	57.6	100.0	100.0	97.3	86.9
8	64.2	56.9	50.4	100.0	96.1	88.1	75.0
9	57.1	50.6	44.8	94.6	86.3	76.7	64.6
10	51.4	45.5	40.3	85.0	76.2	67.2	55.4
11	46.7	41.4	36.6	76.1	67.6	59.3	48.4
12	42.8	37.9	33.6	68.6	61.0	52.0	42.5
13	39.5	35.0	31.0	62.2	55.3	45.9	37.5

[a]Class determined from basic bolt-bearing stress according to Table 7–14.

Table 7–14. *L/D* adjustment class associated with basic bolt-bearing stress

Loading direction	Basic bolt-bearing stress for species group (MPa (lb/in^2))		
	Softwoods	Hardwoods	*L/D* adjustment (Table 7–13)
Parallel	<7.93 (<1,150)	<7.33 (<1,063)	1
	7.93–10.37 (1,150–1,504)	7.33–9.58 (1,063–1,389)	2
	>10.37 (>1,504)	>9.58 (>1,389)	3
Perpendicular	<1.31 (<190)	<1.44 (<209)	1
	1.31–2.00 (190–290)	1.44–2.20 (209–319)	2
	2.00–2.59 (291–375)	2.21–2.84 (320–412)	3
	>2.59 (>375)	>2.84 (>412)	4

by a factor of 0.80 to adjust to wood side plates. The allowable bolt load in pounds is then determined by multiplying by the projected bolt area, *LD*.

Perpendicular to Grain—The starting point for perpendicular-to-grain bolt values is the average green proportional limit stress in compression perpendicular to grain. Procedures in ASTM D2555 are used to establish compression perpendicular values for groups of species. The average proportional limit stress is divided by 1.5 for ring position (growth rings neither parallel nor perpendicular to load during test) and a factor of safety. This value is then multiplied by 1.20 to adjust to a seasoned strength and by 1.10 to adjust to a normal duration of load. The resulting value is called the basic bolt-bearing stress perpendicular to grain.

The basic bolt-bearing stress is then adjusted for the effects of bolt diameter (Table 7–15) and *L/D* ratio (Table 7–13). The allowable bolt load is then determined by multiplying the adjusted basic bolt-bearing stress by the projected bolt area, *LD*.

Post-1991 Yield Model

The empirical design approach used prior to 1991 was based on a tabular value for a single bolt in a wood-to-wood, three-member connection where the side members are each a minimum of one-half the thickness of the main member. The single-bolt value must then be modified for any variation from these reference conditions. The theoretical approach, after 1991, is more general and is not limited to these reference conditions.

The theoretical approach is based on work done in Europe (Johansen 1949) and is referred to as the European Yield Model (EYM). The EYM describes a number of possible yield modes that can occur in a dowel-type connection (Fig. 7–4). The yield strength of these different modes is determined from a static analysis that assumes the wood and the bolt are both perfectly plastic. The yield mode that results in the lowest yield load for a given geometry is the theoretical connection yield load.

Equations corresponding to the yield modes for a three-member joint are given in Table 7–16. (Equations for two-member allowable values are given in the AF&PA *National Design Specification for Wood Construction*) The nominal single-bolt value is dependent on the joint geometry (thickness of main and side members), bolt diameter and bending yield strength, dowel bearing strength, and direction of load to the grain. The equations are equally valid for wood or steel side members, which is taken into account by thickness and dowel bearing strength parameters. The equations are also valid for various load-to-grain directions, which are taken into account by the K_θ and F_e parameter.

The dowel bearing strength is a material property not generally familiar to structural designers. The dowel bearing strength of the wood members is determined from tests that relate species specific gravity and dowel diameter to bearing strength. Empirical equations for these relationships are as follows:

Parallel to grain

$$F_e = 77.2G \quad \text{(metric)} \tag{7–17a}$$

$$F_e = 11{,}200G \quad \text{(inch–pound)} \tag{7–17b}$$

Perpendicular to grain

$$F_e = 212.0G^{1.45}D^{-0.5} \quad \text{(metric)} \tag{7–18a}$$

$$F_e = 6{,}100G^{1.45}D^{-0.5} \quad \text{(inch–pound)} \tag{7–18b}$$

where F_e is dowel bearing strength (MPa, lb/in^2), G specific gravity based on ovendry weight and volume, and D bolt diameter (mm, in.).

Connector Joints

Several types of connectors have been devised that increase joint bearing and shear areas by utilizing rings or plates around bolts holding joint members together. The primary load-carrying portions of these joints are the connectors; the bolts usually serve to prevent transverse separation of the members but do contribute some load-carrying capacity.

The strength of the connector joint depends on the type and size of the connector, the species of wood, the thickness and width of the member, the distance of the connector from the end of the member, the spacing of the connectors, the direction of application of the load with respect to the direction of the grain of the wood, and other factors. Loads for wood joints with steel connectors—split ring (Fig. 7–15) and shear plate (Fig. 7–16)—are discussed in this section. These connectors require closely fitting machined grooves in the wood members.

Parallel-to-Grain Loading

Tests have demonstrated that the density of the wood is a controlling factor in the strength of connector joints. For split-ring connectors, both maximum load and proportional limit load parallel to grain vary linearly with specific gravity (Figs. 7–17 and 7–18). For shear plates, the maximum load and proportional limit load vary linearly with specific gravity for the less dense species (Figs. 7–19 and 7–20). In the higher density species, the shear strength of the bolts becomes the controlling factor. These relations were obtained for seasoned members, approximately 12% moisture content.

Perpendicular-to-Grain Loading

Loads for perpendicular-to-grain loading have been established using three-member joints with the side members loaded parallel to grain. Specific gravity is a good indicator of perpendicular-to-grain strength of timber connector joints. For split-ring connectors, the proportional limit loads perpendicular to grain are 58% of the parallel-to-grain proportional limit loads. The joint deformation at proportional limit is 30% to 50% more than for parallel-to-grain loading.

Table 7–15. Factors for adjusting basic bolt-bearing stress perpendicular to grain for bolt diameter when calculating allowable bolt loads

Bolt diameter (mm (in.))		Adjustment factor
6.35	(1/4)	2.50
9.53	(3/8)	1.95
12.70	(1/2)	1.68
15.88	(5/8)	1.52
19.05	(3/4)	1.41
22.23	(7/8)	1.33
25.40	(1)	1.27
31.75	(1-1/4)	1.19
38.10	(1-1/2)	1.14
44.45	(1-3/4)	1.10
50.80	(2)	1.07
63.50	(2-1/2)	1.03
>76.20	(>3 or over)	1.00

Table 7–16. The 5% offset yield lateral strength (*Z*) for three-member bolted joints

Mode	*Z* value for three-member bolted joint
Mode I_m	$\frac{D t_m F_{em}}{K_\theta}$
Mode I_s	$\frac{2 D t_s F_{es}}{K_\theta}$
Mode III_s	$\frac{2 k_4 D t_s F_{em}}{(2 + R_e) K_\theta}$
Mode IV	$\frac{2D^2}{K_\theta}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$

Definitions

- D nominal bolt diameter, mm (in.)
- F_{em} dowel bearing strength of main (center) member, kPa (lb/in^2)
- F_{es} dowel bearing strength of side members, kPa (lb/in^2)
- F_{yb} bending yield strength of bolt, kPa (lb/in^2)
- K_θ $1 + \theta/360$
- t_m thickness of main (center) member, mm (in.)
- t_s thickness of side member, mm (in.)
- Z nominal single bolt design value
- θ angle of load to grain (degrees)
- R_e $= F_{em}/F_{es}$

$$k_4 = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{2F_{yb}(2+R_e)D^2}{3F_{em}t_s^2}}$$

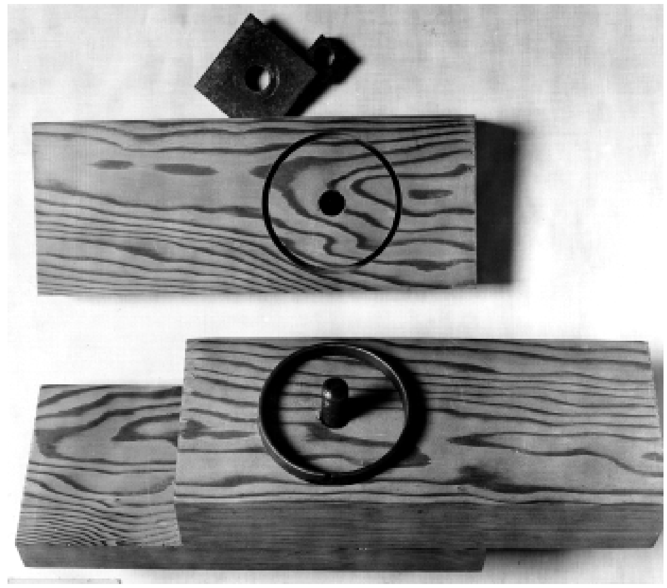

Figure 7–15. Joint with split-ring connector showing connector, precut groove, bolt, washer, and nut.

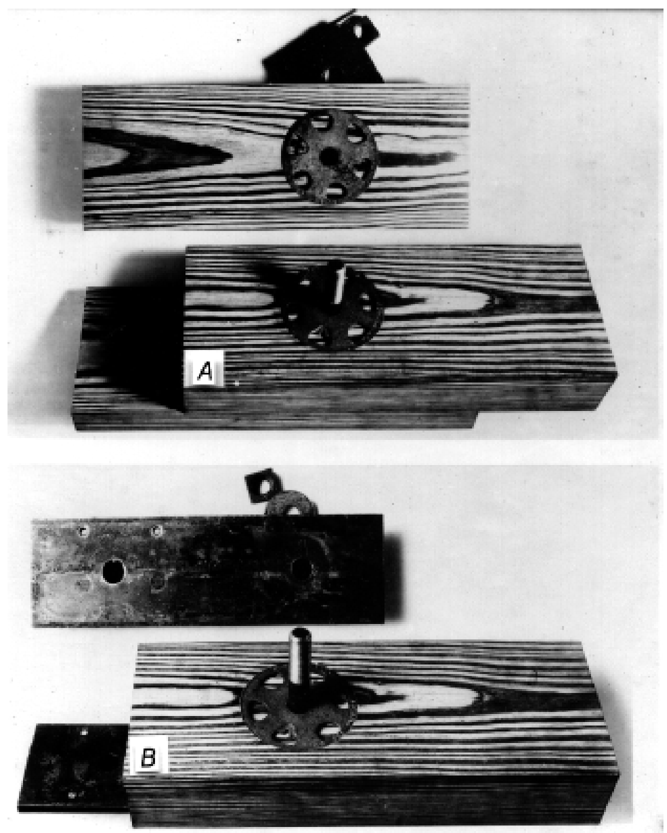

Figure 7–16. Joints with shear-plate connectors with (A) wood side plates and (B) steel side plates.

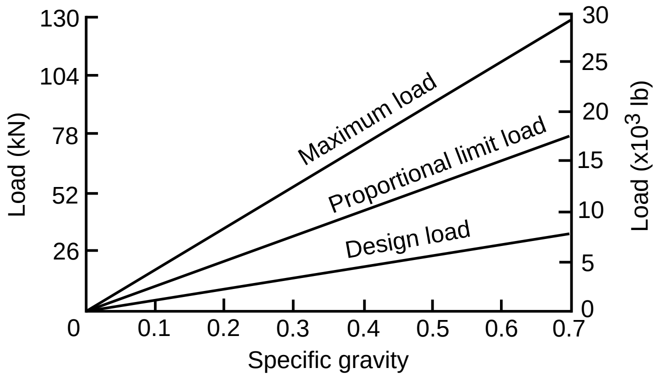

Figure 7–17. Relation between load bearing parallel to grain and specific gravity (ovendry weight, volume at test) for two 63.5-mm (2-1/2-in.) split rings with a single 12.7-mm (1/2-in.) bolt in air-dry material. Center member was thickness 101.6 mm (4 in.) and side member thickness was 50.8 mm (2 in.).

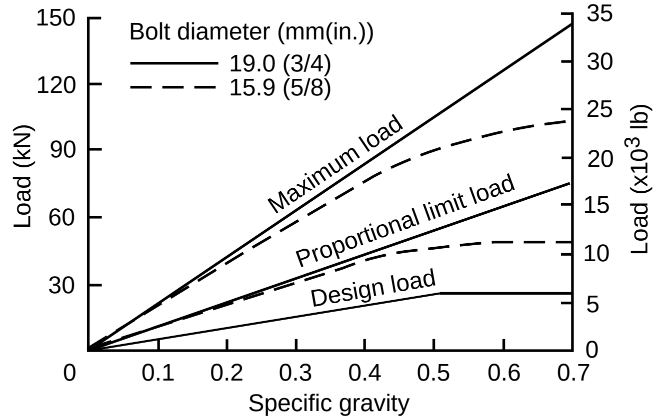

Figure 7–19. Relation between load bearing parallel to grain and specific gravity (ovendry weight, volume at test) for two 66.7-mm (2-5/8-in.) shear plates in air-dry material with steel side plates. Center member thickness was 76.2 mm (3 in.).

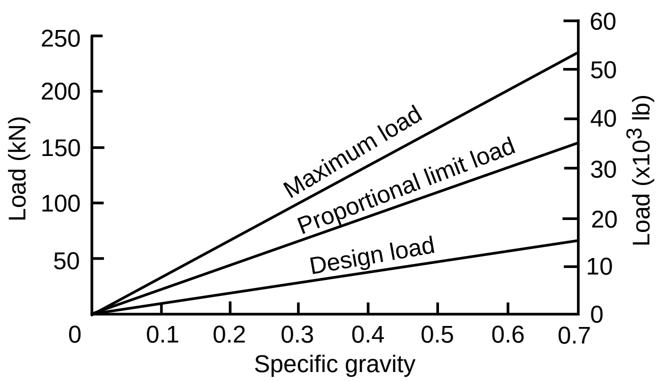

Figure 7–18. Relation between load bearing parallel to grain and specific gravity (ovendry weight, volume at test) for two 101.6-mm (4-in.) split rings and a single 19.1-mm- (3/4-in.-) diameter bolt in air-dry material. Center member thickness was 127.0 mm (5 in.) and side member thickness was 63.5 mm (2-1/2 in.).

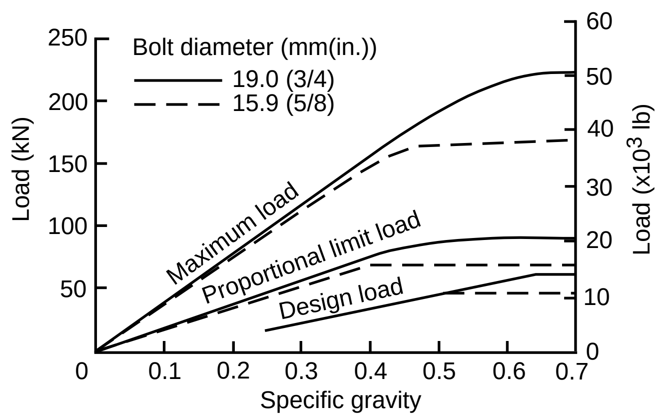

Figure 7–20. Relation between load bearing parallel to grain and specific gravity (ovendry weight, volume at test) for two 101.6-mm (4-in.) shear plates in air-dry material with steel side plates. Center member thickness was 88.9 mm (3-1/2 in.).

For shear-plate connectors, the proportional limit and maximum loads vary linearly with specific gravity (Figs. 7–21 and 7–22). The wood strength controls the joint strength for all species.

Design Loads

Design loads for parallel-to-grain loading have been established by dividing ultimate test loads by an average factor of 4. This gives values that do not exceed five-eighths of the proportional limit loads. The reduction accounts for variability in material, a reduction to long-time loading, and a factor of safety. Design loads for normal duration of load are 10% higher.

For perpendicular-to-grain loading, ultimate load is given less consideration and greater dependence placed on load at proportional limit. For split rings, the proportional limit load is reduced by approximately half. For shear plates, the design loads are approximately five-eighths of the proportional limit test loads. These reductions again account for material variability, a reduction to long-time loading, and a factor of safety.

Design loads are presented in Figures 7–17 to 7–22. In practice, four wood species groups have been established, based primarily on specific gravity, and design loads assigned for each group. Species groupings for connectors are presented in Table 7–17. The corresponding design loads (for long-continued load) are given in Table 7–18. The *National Design Specification for Wood Construction* gives design values for normal-duration load for these and additional species.

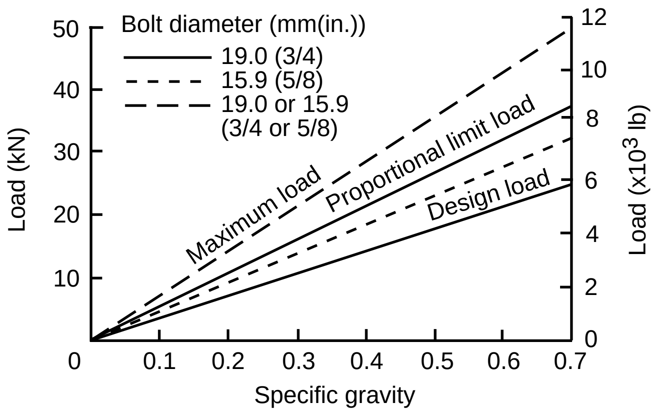

Figure 7–21. Relation between load bearing perpendicular to grain and specific gravity (ovendry weight, volume at test) for two 66.7-mm (2-5/8-in.) shear plates in air-dry material with steel side plates. Center member thickness was 76.2 mm (3 in.).

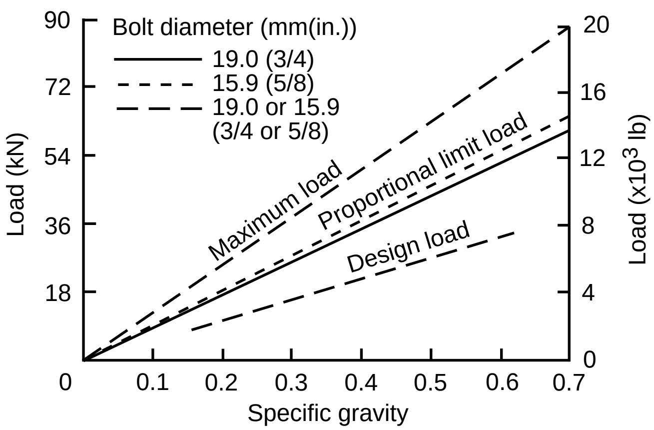

Figure 7–22. Relation between load bearing perpendicular to grain and specific gravity (ovendry weight, volume at test) for two 101.6-mm (4-in.) shear plates in air-dry material with steel side plates. Center member thickness was 88.9 mm (3-1/2 in.).

Modifications

Some factors that affect the loads of connectors were taken into account in deriving the tabular values. Other varied and extreme conditions require modification of the values.

Steel Side Plates

Steel side plates are often used with shear-plate connectors. The loads parallel to grain have been found to be approximately 10% higher than those with wood side plates. The perpendicular-to-grain loads are unchanged.

Exposure and Moisture Condition of Wood

The loads listed in Table 7–18 apply to seasoned members used where they will remain dry. If the wood will be more or less continuously damp or wet in use, two-thirds of the

Table 7–17. Species groupings for connector loads[a]

Connector	Species or species group		
Group 1	Aspen	Basswood	Cottonwood
	Western redcedar	Balsam fir	White fir
	Eastern hemlock	Eastern white pine	Ponderosa pine
	Sugar pine	Western white pine	Engelmann spruce
Group 2	Chestnut	Yellow-poplar	Baldcypress
	Yellow-cedar	Port-Orford-cedar	Western hemlock
	Red pine	Redwood	Red spruce
	Sitka spruce	White spruce	
Group 3	Elm, American	Elm, slippery	Maple, soft
	Sweetgum	Sycamore	Tupelo
	Douglas-fir	Larch, western	Southern Pine
Group 4	Ash, white	Beech	Birch
	Elm, rock	Hickory	Maple, hard
	Oak		

[a]Group 1 woods provide the weakest connector joints; group 4 woods, the strongest.

tabulated values should be used. The amount by which the loads should be reduced to adapt them to other conditions of use depends upon the extent to which the exposure favors decay, the required life of the structure or part, the frequency and thoroughness of inspection, the original cost and the cost of replacements, the proportion of sapwood and durability of the heartwood of the species (if untreated), and the character and efficiency of any treatment. These factors should be evaluated for each individual design. Industry recommendations for the use of connectors when the condition of the lumber is other than continuously wet or continuously dry are given in the *National Design Specification for Wood Construction*.

Ordinarily, before fabrication of connector joints, members should be seasoned to a moisture content corresponding as nearly as practical to that which they will attain in service. This is particularly desirable for lumber for roof trusses and other structural units used in dry locations and in which shrinkage is an important factor. Urgent construction needs sometimes result in the erection of structures and structural units employing green or inadequately seasoned lumber with connectors. Because such lumber subsequently dries out in most buildings, causing shrinkage and opening the joints, adequate maintenance measures must be adopted. The maintenance for connector joints in green lumber should include inspection of the structural units and tightening of all bolts as needed during the time the units are coming to moisture equilibrium, which is normally during the first year.

Table 7–18. Design loads for one connector in a joint[a]

Connector	Minimum thickness of wood member (mm (in.)): With one connector only	Minimum thickness of wood member (mm (in.)): With two connectors in opposite faces, one bolt[b]	Minimum width all members (mm (in.))	Load (N (lb)) Group 1 woods: At 0° angle to grain	Group 1 woods: At 90° angle to grain	Group 2 woods: At 0° angle to grain	Group 2 woods: At 90° angle to grain	Group 3 woods: At 0° angle to grain	Group 3 woods: At 90° angle to grain	Group 4 woods: At 0° angle to grain	Group 4 woods: At 90° angle to grain
Split ring											
63.5-mm (2-1/2-in.) diameter, 19.0 mm (3/4 in.) wide, with 12.7-mm (1/2-in.) bolt	25 (1)	51 (2)	89 (3-1/2)	7,940 (1,785)	4,693 (1,055)	9,274 (2,085)	5,471 (1,230)	11,032 (2,480)	6,561 (1,475)	12,789 (2,875)	7,673 (1,725)
101.6-mm (4-in.) diameter, 25.4 mm (1 in.) wide, with 19.0-mm (3/4-in.) bolt	38 (1-1/2)	76 (3)	140 (5-1/2)	15,324 (3,445)	8,874 (1,995)	17,726 (3,985)	10,275 (2,310)	21,262 (4,780)	12,344 (2,775)	24,821 (5,580)	14,390 (3,235)
Shear plate											
66.7-mm (2-5/8-in.) diameter, 10.7 mm (0.42 in.) wide, with 19.0-mm (3/4-in.) bolt	38 (1-1/2)	67 (2-5/8)	89 (3-1/2)	8,407 (1,890)	4,871 (1,095)	9,742 (2,190)	5,649 (1,270)	11,699 (2,630)	6,784 (1,525)	11,854 (2,665)	7,918 (1,780)
101.6-mm (4-in.) diameter,16.2 mm (0.64 in.) wide, with 19.0-mm or 22.2-mm (3/4- or 7/8-in.) bolt	44 (1-3/4)	92 (3-518)	140 (5-1/2)	12,677 (2,850)	7,362 (1,655)	14,701 (3,305)	8,518 (1,915)	17,637 (3,965)	10,231 (2,300)	20,573 (4,625)	11,943 (2,685)

[a]The loads apply to seasoned timbers in dry, inside locations for a long-continued load. It is also assumed that the joints are properly designed with respect to such features as centering of connectors, adequate end distance, and suitable spacing. Group 1 woods provide the weakest connector joints, group 4 woods the strongest. Species groupings are given in Table 7–17.
[b]A three-member assembly with two connectors takes double the loads indicated.

Grade and Quality of Lumber

The lumber for which the loads for connectors are applicable should conform to the general requirements in regard to quality of structural lumber given in the grading rule books of lumber manufacturers' associations for various commercial species.

The loads for connectors were obtained from tests of joints whose members were clear and free from checks, shakes, and splits. Cross grain at the joint should not be steeper than 1 in 10, and knots in the connector area should be accounted for as explained under Net Section.

Loads at Angle with Grain

The loads for the split-ring and shear-plate connectors for angles of 0° to 90° between direction of load and grain may be obtained by the Hankinson equation (Eq. (7–16)) or by the nomograph in Figure 7–7.

Thickness of Member

The relationship between the loads for the different thicknesses of lumber is based on test results for connector joints. The least thickness of member given in Table 7–18 for the various sizes of connectors is the minimum to obtain optimum load. The loads listed for each type and size of connector are the maximum loads to be used for all thicker lumber. The loads for wood members of thicknesses less than those listed can be obtained by the percentage reductions indicated in Figure 7–23. Thicknesses below those indicated by the curves should not be used.

When one member contains a connector in only one face, loads for thicknesses less than those listed in Table 7–18 can be obtained by the percentage reductions indicated in Figure 7–23 using an assumed thickness equal to twice the actual member thickness.

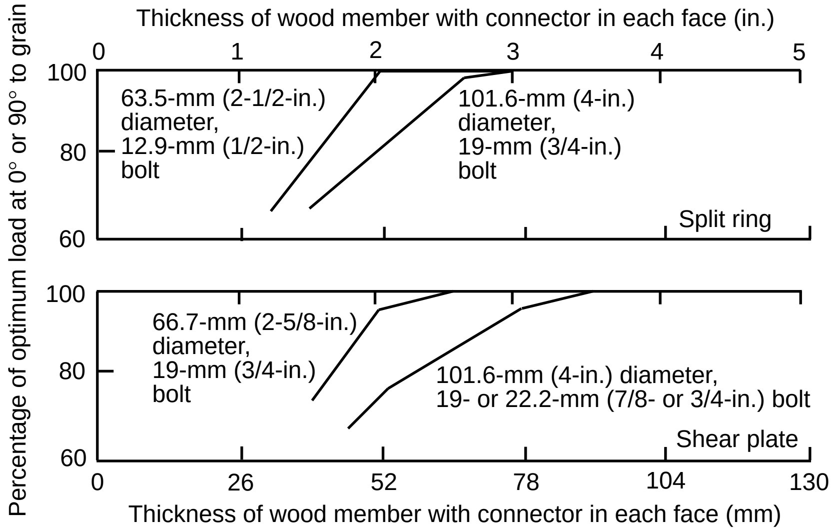

Figure 7–23. Effect of thickness of wood member on the optimum load capacity of a timber connector.

Width of Member

The width of member listed for each type and size of connector is the minimum that should be used. When the connectors are bearing parallel to the grain, no increase in load occurs with an increase in width. When they are bearing perpendicular to the grain, the load increases about 10% for each 25-mm (1-in.) increase in width of member over the minimum widths required for each type and size of connector, up to twice the diameter of the connectors. When the connector is placed off center and the load is applied continuously in one direction only, the proper load can be determined by considering the width of member as equal to twice the edge distance (the distance between the center of the connector and the edge of the member toward which the load is acting). The distance between the center of the connector and the opposite edge should not, however, be less than half the permissible minimum width of the member.

Net Section

The net section is the area remaining at the critical section after subtracting the projected area of the connectors and bolt from the full cross-sectional area of the member. For sawn timbers, the stress in the net area (whether in tension or compression) should not exceed the stress for clear wood in compression parallel to the grain. In using this stress, it is assumed that knots do not occur within a length of half the diameter of the connector from the net section. If knots are present in the longitudinal projection of the net section within a length from the critical section of one-half the diameter of the connector, the area of the knots should be subtracted from the area of the critical section.

In laminated timbers, knots may occur in the inner laminations at the connector location without being apparent from the outside of the member. It is impractical to assure that there are no knots at or near the connector. In laminated construction, therefore, the stress at the net section is limited to the compressive stress for the member, accounting for the effect of knots.

End Distance and Spacing

The load values in Table 7–18 apply when the distance of the connector from the end of the member (end distance *e*) and the spacing *s* between connectors in multiple joints are not factors affecting the strength of the joint (Fig. 7–24A). When the end distance or spacing for connectors bearing parallel to the grain is less than that required to develop the full load, the proper reduced load may be obtained by multiplying the loads in Table 7–18 by the appropriate strength ratio given in Table 7–19. For example, the load for a 102-mm (4-in.) split-ring connector bearing parallel to the grain, when placed 178 mm or more (7 in. or more) from the end of a Douglas-fir tension member that is 38 mm (1-1/2 in.) thick is 21.3 kN (4,780 lb). When the end distance is only 133 mm (5-1/4 in.), the strength ratio obtained by direct interpolation between 178 and 89 mm (7 and 3-1/2 in.) in Table 7–19 is 0.81, and the load equals 0.81 times 21.3 (4,780) or 17.2 kN (3,870 lb).

Placement of Multiple Connectors

Preliminary investigations of the placement of connectors in a multiple-connector joint, together with the observed behavior of single-connector joints tested with variables that simulate those in a multiple-connector joint, are the basis for some suggested design practices.

When two or more connectors in the same face of a member are in a line at right angles to the grain of the member and

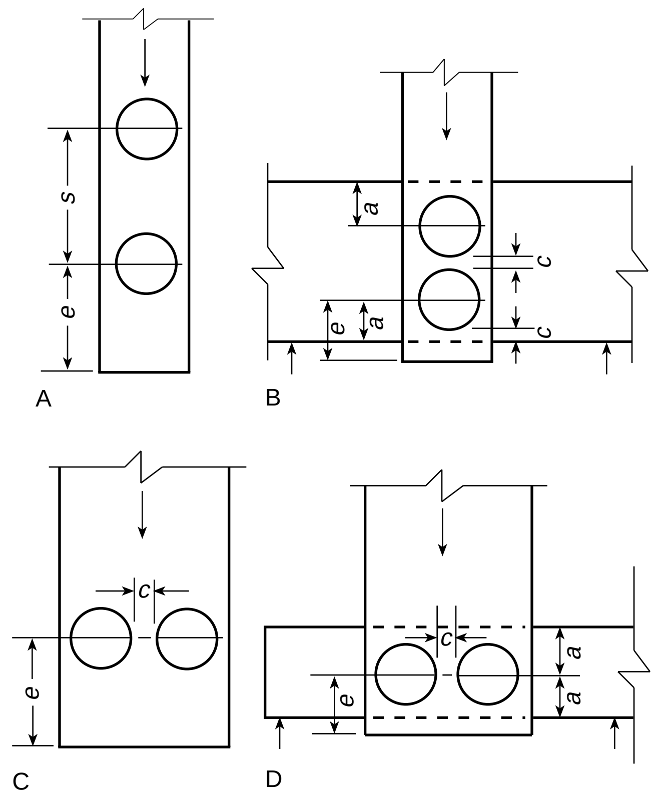

Figure 7–24. Types of multiple-connector joints: A, joint strength depends on end distance *e* and connector spacing *s*; B, joint strength depends on *e*, clear *c*, and edge *a* distances; C, joint strength depends on end *e* and clear *c* distances; D, joint strength depends on end *e*, clear *c*, and edge *a* distances.

are bearing parallel to the grain (Fig. 7–24C), the clear distance *c* between the connectors should not be less than 12.7 mm (1/2 in.). When two or more connectors are acting perpendicular to the grain and are spaced on a line at right angles to the length of the member (Fig. 7–24B), the rules for the width of member and edge distances used with one connector are applicable to the edge distances for multiple connectors. The clear distance *c* between the connectors should be equal to the clear distance from the edge of the member toward which the load is acting to the connector nearest this edge.

In a joint with two or more connectors spaced on a line parallel to the grain and with the load acting perpendicular to the grain (Fig. 7–24D), the available data indicate that the load for multiple connectors is not equal to the sum of the loads for individual connectors. Somewhat more favorable results can be obtained if the connectors are staggered so that they do not act along the same line with respect to the grain of the transverse member. Industry recommendations for various angle-to-grain loadings and spacings are given in the *National Design Specification for Wood Construction*.

Cross Bolts

Cross bolts or stitch bolts placed at or near the end of members joined with connectors or at points between connectors will provide additional safety. They may also be used to reinforce members that have, through change in moisture content in service, developed splits to an undesirable degree.

Multiple-Fastener Joints

When fasteners are used in rows parallel to the direction of loading, total joint load is unequally distributed among fasteners in the row. Simplified methods of analysis have been developed to predict the load distribution among the fasteners in a row. These analyses indicate that the load distribution is a function of (a) the extensional stiffness EA of the joint members, where E is modulus of elasticity and A is gross cross-sectional area, (b) the fastener spacing, (c) the number of fasteners, and (d) the single-fastener load-deformation characteristics.

Theoretically, the two end fasteners carry a majority of the load. For example, in a row of six bolts, the two end bolts will carry more than 50% of the total joint load. Adding bolts to a row tends to reduce the load on the less heavily loaded interior bolts. The most even distribution of bolt loads occurs in a joint where the extensional stiffness of the main member is equal to that of both splice plates. Increasing the fastener spacing tends to put more of the joint load on the end fasteners. Load distribution tends to be worse for stiffer fasteners.

The actual load distribution in field-fabricated joints is difficult to predict. Small misalignment of fasteners, variations in spacing between side and main members, and variations in single-fastener load–deformation characteristics can cause the load distribution to be different than predicted by the theoretical analyses.

For design purposes, modification factors for application to a row of bolts, lag screws, or timber connectors have been developed based on the theoretical analyses. Tables are given in the *National Design Specification for Wood Construction*.

A design equation was developed to replace the double entry required in the *National Design Specification for Wood Construction* tables. This equation was obtained by algebraic simplification of the Lantos analysis that these tables are based on

$$C_g = \left[\frac{m(1-m^{2n})}{n\left[(1+R_{EA}m^n)(1+m)-1+m^{2n}\right]}\right]\left(\frac{1+R_{EA}}{1-m}\right) \qquad (7\text{–}19)$$

where C_g is modification factor, n number of fasteners in a row, R_{EA} the lesser of $(E_sA_s)/(E_mA_m)$ or $(E_mA_m)/(E_sA_s)$, E_m

Table 7–19. Strength ratio for connectors for various longitudinal spacings and end distances[a]

Connector diameter (mm (in.))	Spacing[c] (mm (in.))	Spacing strength ratio	End distance[b] (mm (in.)) Tension member	Compression member	End distance strength ratio
Split-ring					
63.5 (2-1/2)	171.4+ (6-3/4+)	100	139.7+ (5-1/2+)	101.6+ (4+)	100
63.5 (2-1/2)	85.7 (3-3/8)	50	69.8 (2-3/4)	63.5 (2-1/2)	62
101.6 (4)	228.6+ (9+)	100	177.8+ (7+)	139.7+ (5-1/2+)	100
101.6 (4)	123.8 (4-7/8)	50	88.9 (3-1/2)	82.6 (3-1/4)	62
Shear-plate					
66.7 (2-5/8)	171.4+ (6-3/4+)	100	139.7+(5-1/2+)	101.6+ (4+)	100
66.7 (2-5/8)	85.7 (3-3/8)	50	69.8 (2-3/4)	63.5 (2-1/2)	62
101.6 (4)	228.6+ (9+)	100	177.8+ (7+)	139.7+ (5-1/2+)	100
101.6 (4)	114.3 (4-1/2)	50	88.9 (3-1/2)	82.6 (3-1/4)	62

[a]Strength ratio for spacings and end distances intermediate to those listed may be obtained by interpolation and multiplied by the loads in Table 7–18 to obtain design load. The strength ratio applies only to those connector units affected by the respective spacings or end distances. The spacings and end distances should not be less than the minimum shown.
[b]End distance is distance from center of connector to end of member (Fig. 7–24A).
[c]Spacing is distance from center to center of connectors (Fig. 7–24A).

modulus of elasticity of main member, E_s modulus of elasticity of side members, A_m gross cross-sectional area of main member, A_s sum of gross cross-sectional areas of side members, $m = u - \sqrt{u^2 - 1}$, $u = 1 + \gamma(s/2)(1/E_m A_m + 1/E_s A_s)$, s center-to-center spacing between adjacent fasteners in a row, and γ load/slip modulus for a single fastener connection. For 102-mm (4-in.) split-ring or shear-plate connectors,

$$\gamma = 87{,}560 \text{ kN/m} \quad (500{,}000 \text{ lb/in})$$

For 64-mm (2-1/2-in.) split ring or 67-mm (2-5/8-in.) split ring or shear plate connectors,

$$\gamma = 70{,}050 \text{ kN/m} \quad (400{,}000 \text{ lb/in})$$

For bolts or lag screws in wood-to-wood connections,

$$\begin{aligned}\gamma &= 246.25\, D^{1.5} && \text{(metric)}\\ &= 180{,}000\, D^{1.5} && \text{(inch–pound)}\end{aligned}$$

For bolts or lag screws in wood-to-metal connections,

$$\begin{aligned}\gamma &= 369.37\, D^{1.5} && \text{(metric)}\\ &= 270{,}000\, D^{1.5} && \text{(inch–pound)}\end{aligned}$$

where D is diameter of bolt or lag screw.

Metal Plate Connectors

Metal plate connectors, commonly called truss plates, have become a popular means of joining, especially in trussed rafters and joists. These connectors transmit loads by means of teeth, plugs, or nails, which vary from manufacturer to manufacturer. Examples of such plates are shown in Figure 7–25. Plates are usually made of light-gauge galvanized steel and have an area and shape necessary to transmit the forces on the joint. Installation of plates usually requires a hydraulic press or other heavy equipment, although some plates can be installed by hand.

Basic strength values for plate connectors are determined from load–slip curves from tension tests of two butted wood members joined with two plates. Some typical curves are shown in Figure 7–26. Design values are expressed as load per tooth, nail, plug, or unit area of plate. The smallest value as determined by two different means is the design load for normal duration of load: (1) the average load of at least five specimens at 0.38-mm (0.015-in.) slip from plate to wood member or 0.76-mm (0.030-in.) slip from member to member is divided by 1.6; (2) the average ultimate load of at least five specimens is divided by 3.0.

The strength of a metal plate joint may also be controlled by the tensile or shear strength of the plate.

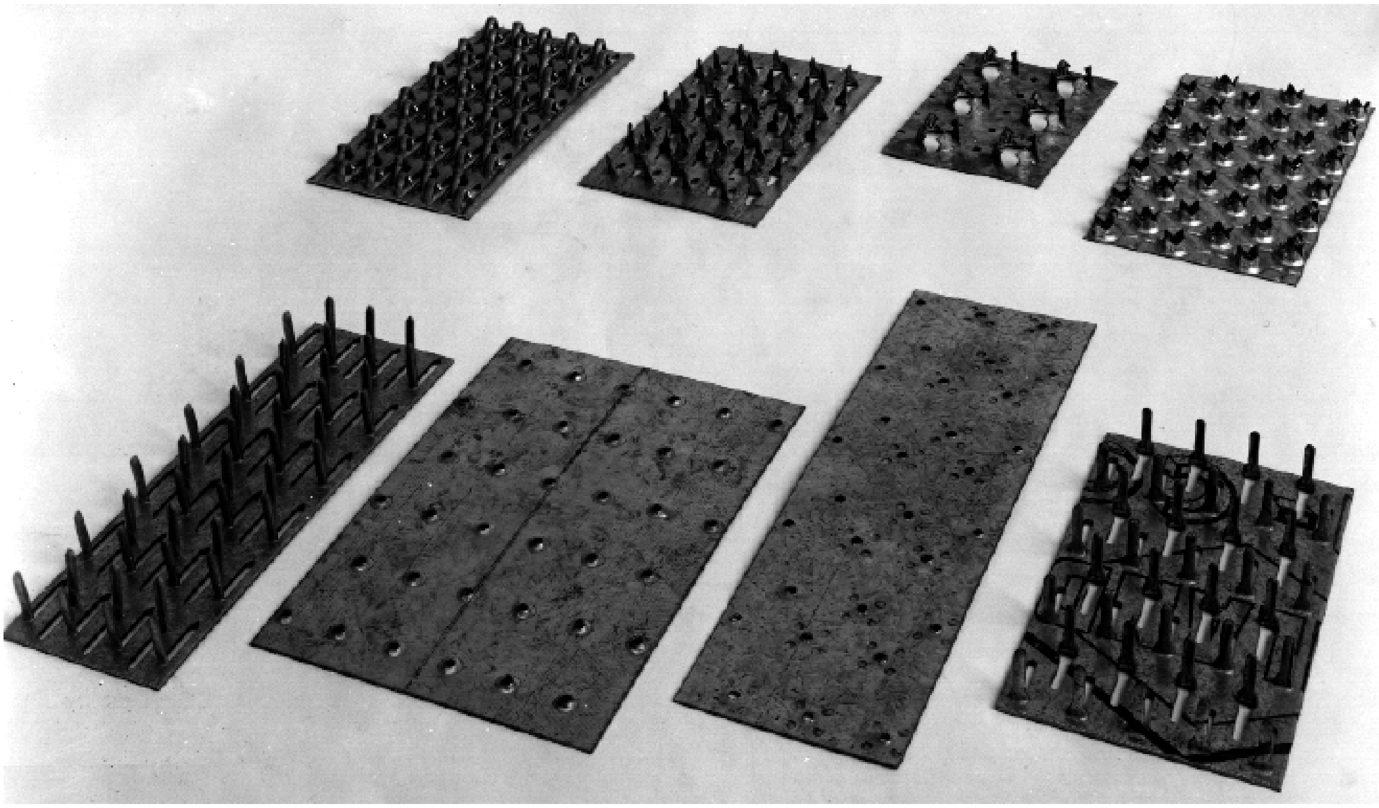

Figure 7–25. Some typical metal plate connectors.

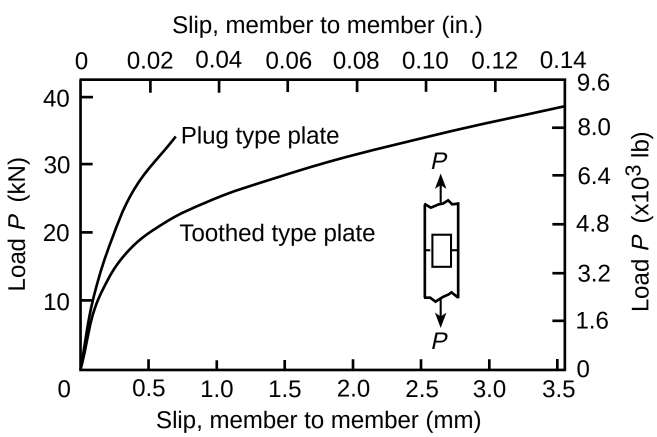

Figure 7–26. Typical load–slip curves for two types of metal plate connectors loaded in tension.

Fastener Head Embedment

The bearing strength of wood under fastener heads is important in such applications as the anchorage of building framework to foundation structures. When pressure tends to pull the framing member away from the foundation, the fastening loads could cause tensile failure of the fastenings, withdrawal of the fastenings from the framing member, or embedment of the fastener heads in the member. The fastener head could even be pulled completely through.

The maximum load for fastener head embedment is related to the fastener head perimeter, while loads at low embedments (1.27 mm (0.05 in.)) are related to the fastener head bearing area. These relations for several species at 10% moisture content are shown in Figures 7–27 and 7–28.

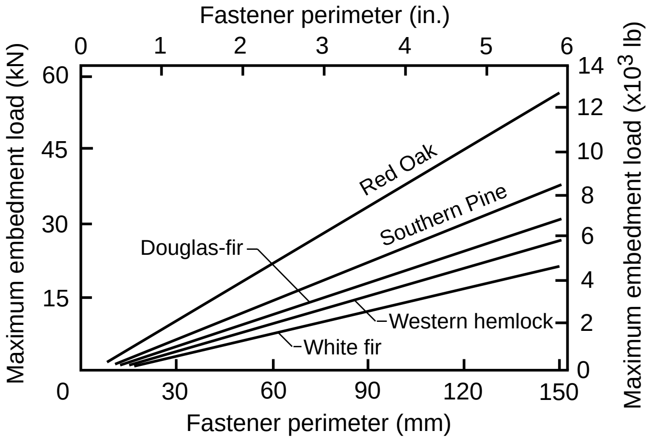

Figure 7–27. Relation between maximum embedment load and fastener perimeter for several species of wood.

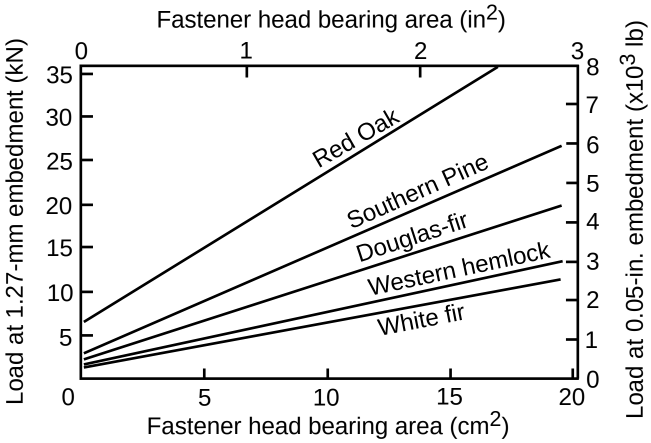

Figure 7–28. Relation between load at 1.27-mm (0.05-in.) embedment and fastener bearing area for several species.

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Chapter 8

Structural Analysis Equations

Lawrence A. Soltis

Contents

Equations for deformation and stress, which are the basis for tension members and beam and column design, are discussed in this chapter. The first two sections cover tapered members, straight members, and special considerations such as notches, slits, and size effect. A third section presents stability criteria for members subject to buckling and for members subject to special conditions. The equations are based on mechanics principles and are not given in the design code format found in Allowable Stress Design or Load and Resistance Factor Design specifications.

Deformation Equations

Equations for deformation of wood members are presented as functions of applied loads, moduli of elasticity and rigidity, and member dimensions. They may be solved to determine minimum required cross-sectional dimensions to meet deformation limitations imposed in design. Average moduli of elasticity and rigidity are given in Chapter 4. Consideration must be given to variability in material properties and uncertainties in applied loads to control reliability of the design.

Axial Load

The deformation of an axially loaded member is not usually an important design consideration. More important considerations will be presented in later sections dealing with combined loads or stability. Axial load produces a change of length given by

$$\delta = \frac{PL}{AE} \qquad (8\text{–}1)$$

where δ is change of length, L length, A cross-sectional area, E modulus of elasticity (E_L when grain runs parallel to member axis), and P axial force parallel to grain.

Bending

Straight Beam Deflection

The deflection of straight beams that are elastically stressed and have a constant cross section throughout their length is given by

$$\delta = \frac{k_b W L^3}{EI} + \frac{k_s W L}{GA'} \qquad (8–2)$$

where δ is deflection, W total beam load acting perpendicular to beam neutral axis, L beam span, k_b and k_s constants dependent upon beam loading, support conditions, and location of point whose deflection is to be calculated, I beam moment of inertia, A' modified beam area, E beam modulus of elasticity (for beams having grain direction parallel to their axis, $E = E_L$), and G beam shear modulus (for beams with flat-grained vertical faces, $G = G_{LT}$, and for beams with edge-grained vertical faces, $G = G_{LR}$). Elastic property values are given in Tables 4–1 and 4–2 (Ch. 4).

The first term on the right side of Equation (8–2) gives the bending deflection and the second term the shear deflection. Values of k_b and k_s for several cases of loading and support are given in Table 8–1.

The moment of inertia I of the beams is given by

$$\begin{aligned} I &= \frac{bh^3}{12} \quad \text{for beam of rectangular cross section} \\ &= \frac{\pi d^4}{64} \quad \text{for beam of circular cross section} \end{aligned} \qquad (8–3)$$

where b is beam width, h beam depth, and d beam diameter. The modified area A' is given by

$$\begin{aligned} A' &= \frac{5}{6} bh \quad \text{for beam of rectangular cross section} \\ &= \frac{9}{40} \pi d^2 \quad \text{for beam of circular cross section} \end{aligned} \qquad (8–4)$$

If the beam has initial deformations such as bow (lateral bend) or twist, these deformations will be increased by the bending loads. It may be necessary to provide lateral or torsional restraints to hold such members in line. (See Interaction of Buckling Modes section.)

Tapered Beam Deflection

Figures 8–1 and 8–2 are useful in the design of tapered beams. The ordinates are based on design criteria such as span, loading, difference in beam height ($h_c - h_0$) as required by roof slope or architectural effect, and maximum allowable deflection, together with material properties. From this, the value of the abscissa can be determined and the smallest beam depth h_0 can be calculated for comparison with that given by the design criteria. Conversely, the deflection of a beam can be calculated if the value of the abscissa is known. Tapered beams deflect as a result of shear deflection in addition to bending deflections (Figs. 8–1 and 8–2), and this shear deflection Δ_s can be closely approximated by

$$\begin{aligned} \Delta_s &= \frac{3WL}{20Gbh_0} \quad \text{for uniformly distributed load} \\ &= \frac{3PL}{10Gbh_0} \quad \text{for midspan-concentrated load} \end{aligned} \qquad (8–5)$$

The final beam design should consider the total deflection as the sum of the shear and bending deflection, and it may be necessary to iterate to arrive at final beam dimensions. Equations (8–5) are applicable to either single-tapered or double-tapered beams. As with straight beams, lateral or torsional restraint may be necessary.

Effect of Notches and Holes

The deflection of beams is increased if reductions in cross-section dimensions occur, such as by holes or notches. The deflection of such beams can be determined by considering them of variable cross section along their length and appropriately solving the general differential equations of the elastic curves, $EI(d^2y/dx^2) = M$, to obtain deflection expressions or by the application of Castigliano's theorem. (These procedures are given in most texts on strength of materials.)

Table 8–1. Values of k_b and k_s for several beam loadings

Loading	Beam ends	Deflection at	k_b	k_s
Uniformly distributed	Both simply supported	Midspan	5/384	1/8
	Both clamped	Midspan	1/384	1/8
Concentrated at midspan	Both simply supported	Midspan	1/48	1/4
	Both clamped	Midspan	1/192	1/4
Concentrated at outer quarter span points	Both simply supported	Midspan	11/768	1/8
	Both simply supported	Load point	1/96	1/8
Uniformly distributed	Cantilever, one free, one clamped	Free end	1/8	1/2
Concentrated at free end	Cantilever, one free, one clamped	Free end	1/3	1

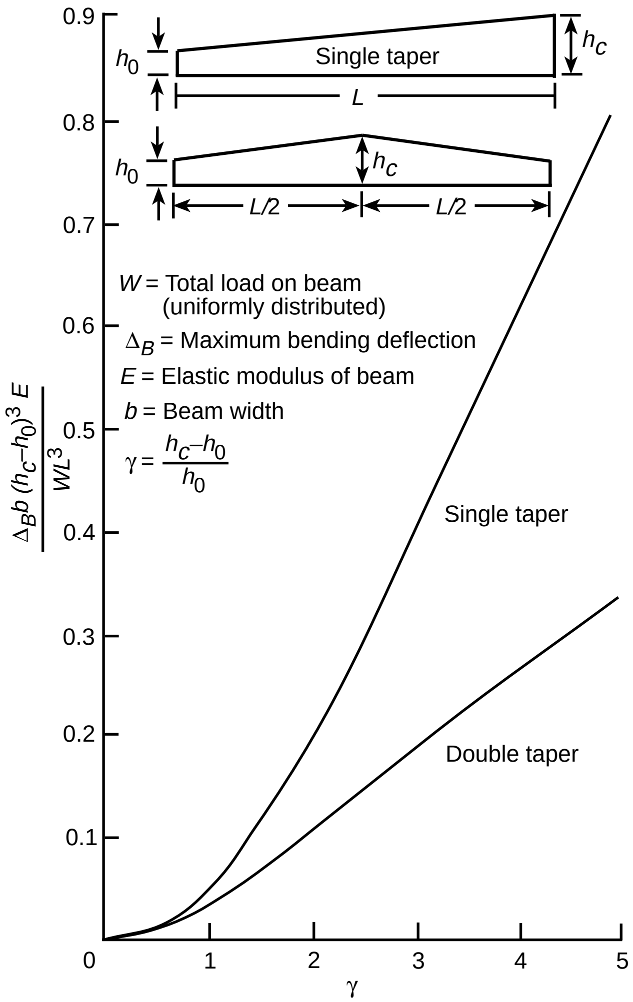

Figure 8–1. Graph for determining tapered beam size based on deflection under uniformly distributed load.

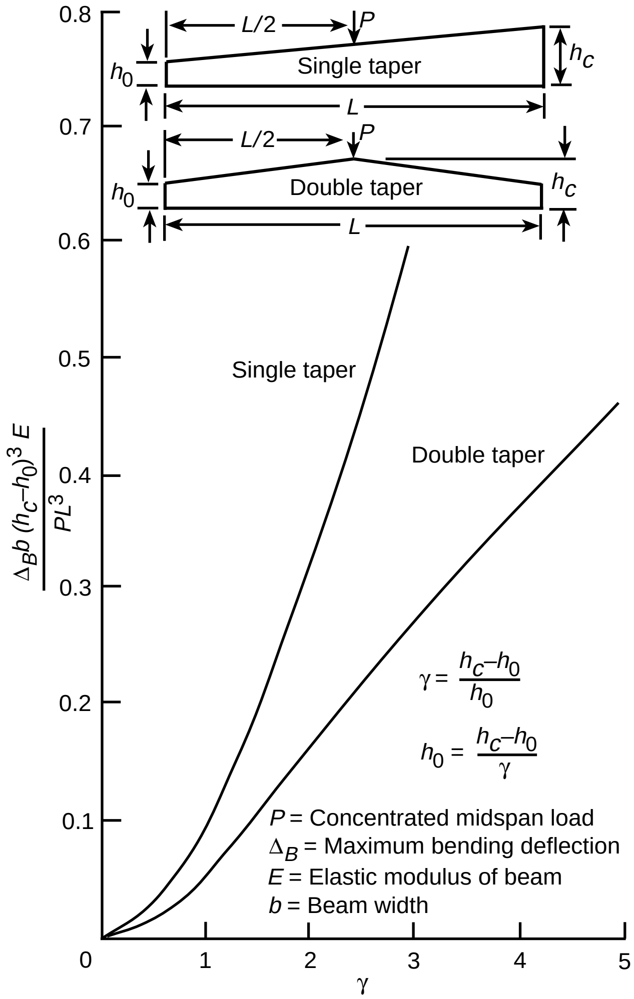

Figure 8–2. Graph for determining tapered beam size on deflection under concentrated midspan load.

Effect of Time: Creep Deflections

In addition to the elastic deflections previously discussed, wood beams usually sag in time; that is, the deflection increases beyond what it was immediately after the load was first applied. (See the discussion of creep in Time Under Load in Ch. 4.)

Green timbers, in particular, will sag if allowed to dry under load, although partially dried material will also sag to some extent. In thoroughly dried beams, small changes in deflection occur with changes in moisture content but with little permanent increase in deflection. If deflection under longtime load with initially green timber is to be limited, it has been customary to design for an initial deflection of about half the value permitted for longtime deflection. If deflection under longtime load with initially dry timber is to be limited, it has been customary to design for an initial deflection of about two-thirds the value permitted for longtime deflection.

Water Ponding

Ponding of water on roofs already deflected by other loads can cause large increases in deflection. The total deflection Δ due to design load plus ponded water can be closely estimated by

$$\Delta = \frac{\Delta_0}{1 - S/S_{cr}} \tag{8–6}$$

where Δ_0 is deflection due to design load alone, S beam spacing, and S_{cr} critical beam spacing (Eq. (8–31)).

Combined Bending and Axial Load

Concentric Load

Addition of a concentric axial load to a beam under loads acting perpendicular to the beam neutral axis causes increase in bending deflection for added axial compression and decrease in bending deflection for added axial tension.

The deflection under combined loading at midspan for pin-ended members can be estimated closely by

$$\Delta = \frac{\Delta_0}{1 \pm P/P_{\mathrm{cr}}} \qquad (8\text{–}7)$$

where the plus sign is chosen if the axial load is tension and the minus sign if the axial load is compression, Δ is midspan deflection under combined loading, Δ_0 beam midspan deflection without axial load, P axial load, and P_{cr} a constant equal to the buckling load of the beam under axial compressive load only (see Axial Compression in Stability Equations section.) based on flexural rigidity about the neutral axis perpendicular to the direction of bending loads. This constant appears regardless of whether P is tension or compression. If P is compression, it must be less than P_{cr} to avoid collapse. When the axial load is tension, it is conservative to ignore the P/P_{cr} term. (If the beam is not supported against lateral deflection, its buckling load should be checked using Eq. (8–35).)

Eccentric Load

If an axial load is eccentrically applied to a pin-ended member, it will induce bending deflections and change in length given by Equation (8–1). Equation (8–7) can be applied to find the bending deflection by writing the equation in the form

$$\delta_{\mathrm{b}} + \varepsilon_0 = \frac{\varepsilon_0}{1 \pm P/P_{\mathrm{cr}}} \qquad (8\text{–}8)$$

where δ_{b} is the induced bending deflection at midspan and ε_0 the eccentricity of P from the centroid of the cross section.

Torsion

The angle of twist of wood members about the longitudinal axis can be computed by

$$\theta = \frac{TL}{GK} \qquad (8\text{–}9)$$

where θ is angle of twist in radians, T applied torque, L member length, G shear modulus (use $\sqrt{G_{LR}G_{LT}}$, or approximate G by $E_L/16$ if measured G is not available), and K a cross-section shape factor. For a circular cross section, K is the polar moment of inertia:

$$K = \frac{\pi D^4}{32} \qquad (8\text{–}10)$$

where D is diameter. For a rectangular cross section,

$$K = \frac{hb^3}{\phi} \qquad (8\text{–}11)$$

where h is larger cross-section dimension, b is smaller cross-section dimension, and ϕ is given in Figure 8–3.

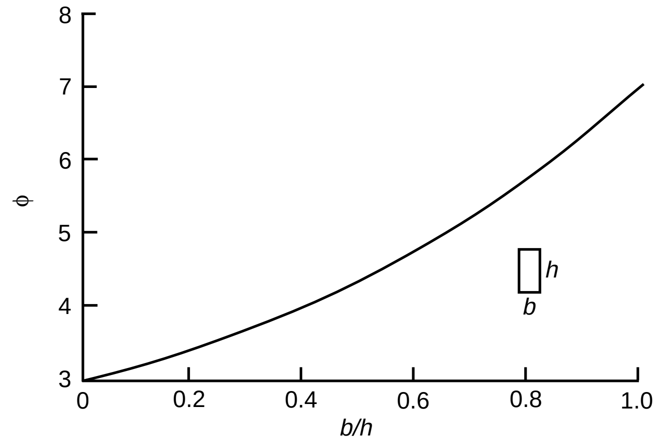

Figure 8–3. Coefficient ϕ for determining torsional rigidity of rectangular member (Eq. (8 –11)).

Stress Equations

The equations presented here are limited by the assumption that stress and strain are directly proportional (Hooke's law) and by the fact that local stresses in the vicinity of points of support or points of load application are correct only to the extent of being statically equivalent to the true stress distribution (St. Venant's principle). Local stress concentrations must be separately accounted for if they are to be limited in design.

Axial Load

Tensile Stress

Concentric axial load (along the line joining the centroids of the cross sections) produces a uniform stress:

$$f_{\mathrm{t}} = \frac{P}{A} \qquad (8\text{–}12)$$

where f_{t} is tensile stress, P axial load, and A cross-sectional area.

Short-Block Compressive Stress

Equation (8–12) can also be used in compression if the member is short enough to fail by simple crushing without deflecting laterally. Such fiber crushing produces a local "wrinkle" caused by microstructural instability. The member as a whole remains structurally stable and able to bear load.

Bending

The strength of beams is determined by flexural stresses caused by bending moment, shear stresses caused by shear load, and compression across the grain at the end bearings and load points.

Straight Beam Stresses

The stress due to bending moment for a simply supported pin-ended beam is a maximum at the top and bottom edges. The concave edge is compressed, and the convex edge is under tension. The maximum stress is given by

$$f_b = \frac{M}{Z} \tag{8–13}$$

where f_b is bending stress, M bending moment, and Z beam section modulus (for a rectangular cross section, $Z = bh^2/6$; for a circular cross section, $Z = \pi D^3/32$).

This equation is also used beyond the limits of Hooke's law with M as the ultimate moment at failure. The resulting pseudo-stress is called the "modulus of rupture," values of which are tabulated in Chapter 4. The modulus of rupture has been found to decrease with increasing size of member. (See Size Effect section.)

The shear stress due to bending is a maximum at the centroidal axis of the beam, where the bending stress happens to be zero. (This statement is not true if the beam is tapered—see following section.) In wood beams this shear stress may produce a failure crack near mid-depth running along the axis of the member. Unless the beam is sufficiently short and deep, it will fail in bending before shear failure can develop; but wood beams are relatively weak in shear, and shear strength can sometimes govern a design. The maximum shear stress is

$$f_s = k\frac{V}{A} \tag{8–14}$$

where f_s is shear stress, V vertical shear force on cross section, A cross-sectional area, and $k = 3/2$ for a rectangular cross section or $k = 4/3$ for a circular cross section.

Tapered Beam Stresses

For beams of constant width that taper in depth at a slope less than 25°, the bending stress can be obtained from Equation (8–13) with an error of less than 5%. The shear stress, however, differs markedly from that found in uniform beams. It can be determined from the basic theory presented by Maki and Kuenzi (1965). The shear stress at the tapered edge can reach a maximum value as great as that at the neutral axis at a reaction.

Consider the example shown in Figure 8–4, in which concentrated loads farther to the right have produced a support reaction V at the left end. In this case the maximum stresses occur at the cross section that is double the depth of the beam at the reaction. For other loadings, the location of the cross section with maximum shear stress at the tapered edge will be different.

For the beam depicted in Figure 8–4, the bending stress is also a maximum at the same cross section where the shear stress is maximum at the tapered edge. This stress situation also causes a stress in the direction perpendicular to the neutral axis that is maximum at the tapered edge. The effect of combined stresses at a point can be approximately accounted for by an interaction equation based on the Henky–von Mises theory of energy due to the change of shape. This theory applied by Norris (1950) to wood results in

$$\frac{f_x^2}{F_x^2} + \frac{f_{xy}^2}{F_{xy}^2} + \frac{f_y^2}{F_y^2} = 1 \tag{8–15}$$

where f_x is bending stress, f_y stress perpendicular to the neutral axis, and f_{xy} shear stress. Values of F_x, F_y, and F_{xy} are corresponding stresses chosen at design values or maximum values in accordance with allowable or maximum values being determined for the tapered beam. Maximum stresses in

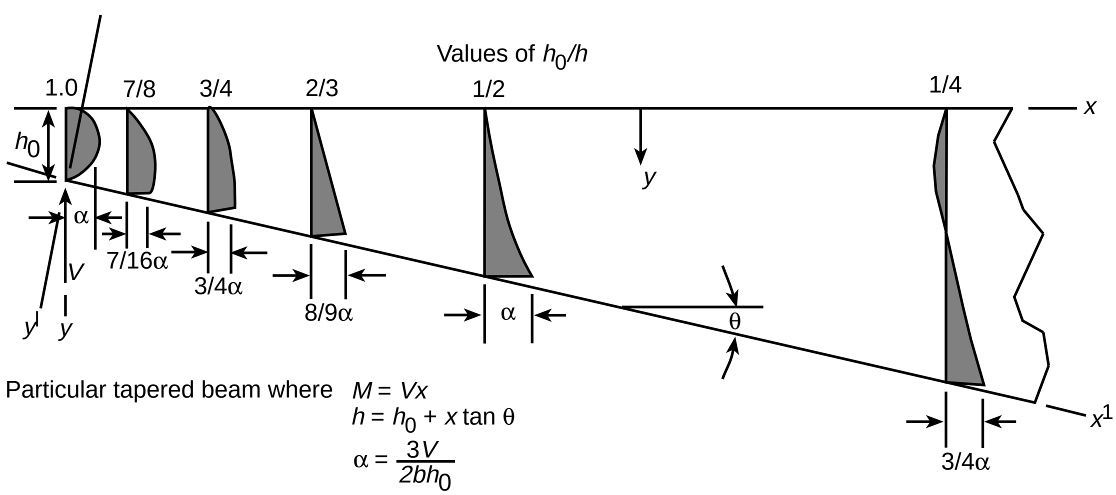

Figure 8–4. Shear stress distribution for a tapered beam.

the beam depicted in Figure 8–4 are given by

$$f_x = \frac{3M}{2bh_0^2}$$
$$f_{xy} = f_x \tan\theta \qquad (8\text{–}16)$$
$$f_y = f_x \tan^2\theta$$

Substitution of these equations into the interaction Equation (8–15) will result in an expression for the moment capacity M of the beam. If the taper is on the beam tension edge, the values of f_x and f_y are tensile stresses.

Example: Determine the moment capacity (newton-meters) of a tapered beam of width b = 100 mm, depth h_0 = 200 mm, and taper tan θ = 1/10. Substituting these dimensions into Equation (8–16) (with stresses in pascals) results in

$$f_x = 375M$$
$$f_{xy} = 37.5M$$
$$f_y = 3.75M$$

Substituting these into Equation (8–15) and solving for M results in

$$M = \frac{1}{3.75\left[10^4/F_x^2 + 10^2/F_{xy}^2 + 1/F_y^2\right]^{1/2}}$$

where appropriate allowable or maximum values of the F stresses (pascals) are chosen.

Size Effect

The modulus of rupture (maximum bending stress) of wood beams depends on beam size and method of loading, and the strength of clear, straight-grained beams decreases as size increases. These effects were found to be describable by statistical strength theory involving "weakest link" hypotheses and can be summarized as follows: For two beams under two equal concentrated loads applied symmetrical to the midspan points, the ratio of the modulus of rupture of beam 1 to the modulus of rupture of beam 2 is given by

$$\frac{R_1}{R_2} = \left[\frac{h_2L_2(1+ma_2/L_2)}{h_1L_1(1+ma_1/L_1)}\right]^{1/m} \qquad (8\text{–}17)$$

where subscripts 1 and 2 refer to beam 1 and beam 2, R is modulus of rupture, h beam depth, L beam span, a distance between loads placed a/2 each side of midspan, and m a constant. For clear, straight-grained Douglas-fir beams, m = 18. If Equation (8–17) is used for beam 2 size (Ch. 4) loaded at midspan, then h_2 = 5.08 mm (2 in.), L_2 = 71.112 mm (28 in.), and a_2 = 0 and Equation (8–17) becomes

$$\frac{R_1}{R_2} = \left[\frac{361.29}{h_1L_1(1+ma_1/L_1)}\right]^{1/m} \qquad \text{(metric)} \qquad (8\text{–}18a)$$

$$\frac{R_1}{R_2} = \left[\frac{56}{h_1L_1(1+ma_1/L_1)}\right]^{1/m} \qquad \text{(inch–pound)} \qquad (8\text{–}18b)$$

Example: Determine modulus of rupture for a beam 10 in. deep, spanning 18 ft, and loaded at one-third span points compared with a beam 2 in. deep, spanning 28 in., and loaded at midspan that had a modulus of rupture of 10,000 lb/in^2. Assume m = 18. Substituting the dimensions into Equation (8–18) produces

$$R_1 = 10{,}000\left[\frac{56}{2{,}160(1+6)}\right]^{1/18}$$
$$= 7{,}330 \text{ lb/in}^2$$

Application of the statistical strength theory to beams under uniformly distributed load resulted in the following relationship between modulus of rupture of beams under uniformly distributed load and modulus of rupture of beams under concentrated loads:

$$\frac{R_u}{R_c} = \left[\frac{(1+18a_c/L_c)h_cL_c}{3.876h_uL_u}\right]^{1/18} \qquad (8\text{–}19)$$

where subscripts u and c refer to beams under uniformly distributed and concentrated loads, respectively, and other terms are as previously defined.

Shear strength for non-split, non-checked, solid-sawn, and glulam beams also decreases as beam size increases. A relationship between beam shear τ and ASTM shear block strength τ_{ASTM}, including a stress concentration factor for the re-entrant corner of the shear block, C_f, and the shear area A, is

$$\tau = \frac{1.9C_f\tau_{ASTM}}{A^{1/5}} \qquad \text{(metric)} \qquad (8\text{–}20a)$$

$$\tau = \frac{1.3C_f\tau_{ASTM}}{A^{1/5}} \qquad \text{(inch–pound)} \qquad (8\text{–}20b)$$

where τ is beam shear (MPa, lb/in^2), C_f stress concentration factor, τ_{ASTM} ASTM shear block strength (MPa, lb/in^2), and A shear area (cm^2, in^2).

This relationship was determined by empirical fit to test data. The shear block re-entrant corner concentration factor is approximately 2; the shear area is defined as beam width multiplied by the length of beam subjected to shear force.

Effect of Notches, Slits, and Holes

In beams having notches, slits, or holes with sharp interior corners, large stress concentrations exist at the corners. The local stresses include shear parallel to grain and tension

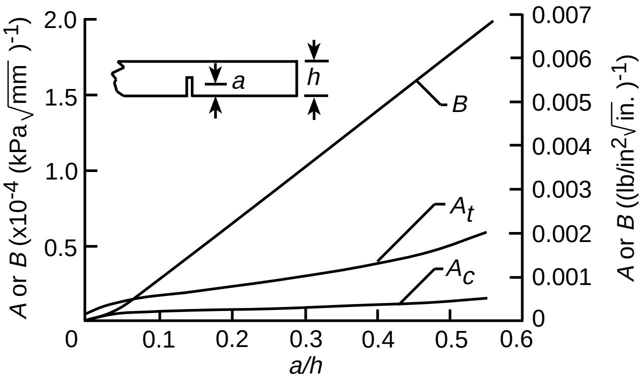

Figure 8–5. Coefficients *A* and *B* for crack-initiation criterion (Eq. (8–21)).

perpendicular to grain. As a result, even moderately low loads can cause a crack to initiate at the sharp corner and propagate along the grain. An estimate of the crack-initiation load can be obtained by the fracture mechanics analysis of Murphy (1979) for a beam with a slit, but it is generally more economical to avoid sharp notches entirely in wood beams, especially large wood beams, since there is a size effect: sharp notches cause greater reductions in strength for larger beams. A conservative criterion for crack initiation for a beam with a slit is

$$\sqrt{h}\left[A\left(\frac{6M}{bh^2}\right)+B\left(\frac{3V}{2bh}\right)\right]=1 \qquad (8\text{–}21)$$

where h is beam depth, b beam width, M bending moment, and V vertical shear force, and coefficients A and B are presented in Figure 8–5 as functions of a/h, where a is slit depth. The value of A depends on whether the slit is on the tension edge or the compression edge. Therefore, use either A_t or A_c as appropriate. The values of A and B are dependent upon species; however, the values given in Figure 8–5 are conservative for most softwood species.

Effects of Time: Creep Rupture, Fatigue, and Aging

See Chapter 4 for a discussion of fatigue and aging. Creep rupture is accounted for by duration-of-load adjustment in the setting of allowable stresses, as discussed in Chapters 4 and 6.

Water Ponding

Ponding of water on roofs can cause increases in bending stresses that can be computed by the same amplification factor (Eq. (8–6)) used with deflection. (See Water Ponding in the Deformation Equations section.)

Combined Bending and Axial Load

Concentric Load

Equation (8–7) gives the effect on deflection of adding an end load to a simply supported pin-ended beam already bent by transverse loads. The bending stress in the member is modified by the same factor as the deflection:

$$f_b = \frac{f_{b0}}{1 \pm P/P_{cr}} \qquad (8\text{–}22)$$

where the plus sign is chosen if the axial load is tension and the minus sign is chosen if the axial load is compression, f_b is net bending stress from combined bending and axial load, f_{b0} bending stress without axial load, P axial load, and P_{cr} the buckling load of the beam under axial compressive load only (see Axial Compression in the Stability Equations section), based on flexural rigidity about the neutral axis perpendicular to the direction of the bending loads. This P_{cr} is not necessarily the minimum buckling load of the member. If P is compressive, the possibility of buckling under combined loading must be checked. (See Interaction of Buckling Modes.)

The total stress under combined bending and axial load is obtained by superposition of the stresses given by Equations (8–12) and (8–22).

Example: Suppose transverse loads produce a bending stress f_{b0} tensile on the convex edge and compressive on the concave edge of the beam. Then the addition of a tensile axial force P at the centroids of the end sections will produce a maximum tensile stress on the convex edge of

$$f_{t\,max} = \frac{f_{b0}}{1+P/P_{cr}} + \frac{P}{A}$$

and a maximum compressive stress on the concave edge of

$$f_{c\,max} = \frac{f_{b0}}{1+P/P_{cr}} - \frac{P}{A}$$

where a negative result would indicate that the stress was in fact tensile.

Eccentric Load

If the axial load is eccentrically applied, then the bending stress f_{b0} should be augmented by $\pm P\varepsilon_0/Z$, where ε_0 is eccentricity of the axial load.

Example: In the preceding example, let the axial load be eccentric toward the concave edge of the beam. Then the maximum stresses become

$$f_{t\,max} = \frac{f_{b0} - P\varepsilon_0/Z}{1+P/P_{cr}} + \frac{P}{A}$$

$$f_{c\,max} = \frac{f_{b0} - P\varepsilon_0/Z}{1+P/P_{cr}} - \frac{P}{A}$$

Torsion

For a circular cross section, the shear stress induced by torsion is

$$f_s = \frac{16T}{\pi D^3} \qquad (8\text{–}23)$$

where T is applied torque and D diameter. For a rectangular cross section,

$$f_s = \frac{T}{\beta h b^2} \qquad (8\text{–}24)$$

where T is applied torque, h larger cross-section dimension, and b smaller cross-section dimension, and β is presented in Figure 8–6.

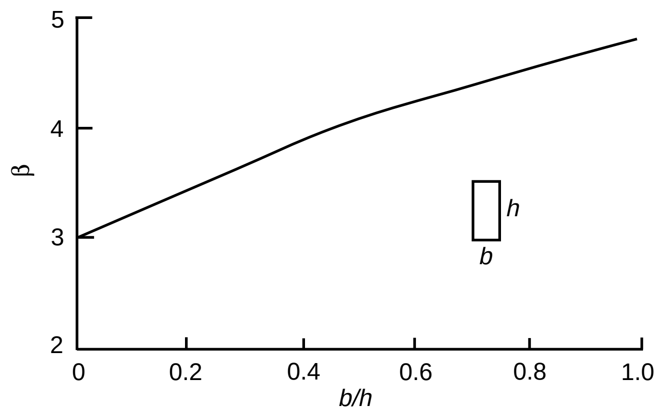

Figure 8–6. Coefficient β for computing maximum shear stress in torsion of rectangular member (Eq. (8 –24)).

Stability Equations

Axial Compression

For slender members under axial compression, stability is the principal design criterion. The following equations are for concentrically loaded members. For eccentrically loaded columns, see Interaction of Buckling Modes section.

Long Columns

A column long enough to buckle before the compressive stress P/A exceeds the proportional limit stress is called a "long column." The critical stress at buckling is calculated by Euler's formula:

$$f_{cr} = \frac{\pi^2 E_L}{(L/r)^2} \qquad (8\text{–}25)$$

where E_L is elastic modulus parallel to the axis of the member, L unbraced length, and r least radius of gyration (for a rectangular cross section with b as its least dimension, $r = b/\sqrt{12}$, and for a circular cross section, $r = d/4$). Equation (8–25) is based on a pinned-end condition but may be used conservatively for square ends as well.

Short Columns

Columns that buckle at a compressive stress P/A beyond the proportional limit stress are called "short columns." Usually the short column range is explored empirically, and appropriate design equations are proposed. Material of this nature is presented in *USDA Technical Bulletin 167* (Newlin and Gahagan 1930). The final equation is a fourth-power parabolic function that can be written as

$$f_{cr} = F_c\left[1 - \frac{4}{27\pi^4}\left(\frac{L}{r}\sqrt{\frac{F_c}{E_L}}\right)^4\right] \qquad (8\text{–}26)$$

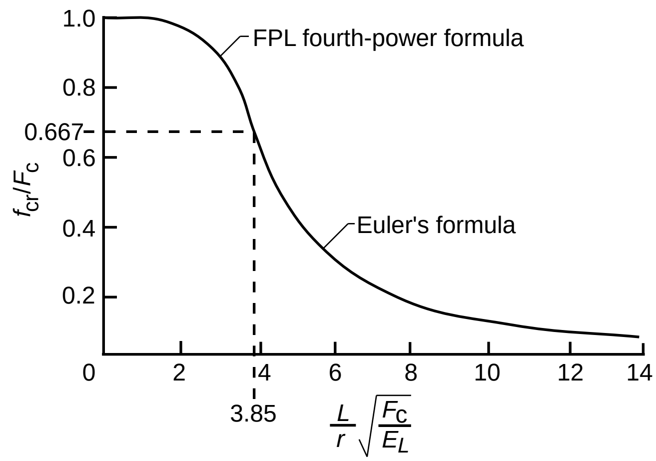

Figure 8–7. Graph for determining critical buckling stress of wood columns.

where F_c is compressive strength and remaining terms are defined as in Equation (8–25). Figure 8–7 is a graphical representation of Equations (8–25) and (8–26).

Short columns can be analyzed by fitting a nonlinear function to compressive stress–strain data and using it in place of Hooke's law. One such nonlinear function proposed by Ylinen (1956) is

$$\varepsilon = \frac{F_c}{E_L}\left[c\frac{f}{F_c} - (1-c)\log_e\left(1 - \frac{f}{F_c}\right)\right] \qquad (8\text{–}27)$$

where ε is compressive strain, f compressive stress, c a constant between 0 and 1, and E_L and F_c are as previously defined. Using the slope of Equation (8–27) in place of E_L in Euler's formula (Eq. (8–25)) leads to Ylinen's buckling equation

$$f_{cr} = \frac{F_c + f_e}{2c} - \sqrt{\left(\frac{F_c + f_e}{2c}\right)^2 - \frac{F_c f_e}{c}} \qquad (8\text{–}28)$$

where F_c is compressive strength and f_e buckling stress given by Euler's formula (Eq. (8–25)). Equation (8–28) can be made to agree closely with Figure 8–7 by choosing $c = 0.957$.

Comparing the fourth-power parabolic function Equation (8–26) to experimental data indicates the function is nonconservative for intermediate L/r range columns. Using Ylinen's buckling equation with $c = 0.8$ results in a better approximation of the solid-sawn and glued-laminated data.

Built-Up and Spaced Columns

Built-up columns of nearly square cross section with the lumber nailed or bolted together will not support loads as great as if the lumber were glued together. The reason is that shear distortions can occur in the mechanical joints.

If built-up columns are adequately connected and the axial load is near the geometric center of the cross section, Equation (8–28) is reduced with a factor that depends on the type of mechanical connection. The built-up column capacity is

$$f_{cr} = K_f\left[\frac{F_c + f_e}{2c} - \sqrt{\left(\frac{F_c + f_e}{2c}\right)^2 - \frac{F_c f_e}{c}}\right] \qquad (8\text{–}29)$$

where F_c, f_e, and c are as defined for Equation (8–28). K_f is the built-up stability factor, which accounts for the efficiency of the connection; for bolts, $K_f = 0.75$, and for nails, $K_f = 0.6$, provided bolt and nail spacing requirements meet design specification approval.

If the built-up column is of several spaced pieces, the spacer blocks should be placed close enough together, lengthwise in the column, so that the unsupported portion of the spaced member will not buckle at the same or lower stress than that of the complete member. "Spaced columns" are designed with previously presented column equations, considering each compression member as an unsupported simple column; the sum of column loads for all the members is taken as the column load for the spaced column.

Columns With Flanges

Columns with thin, outstanding flanges can fail by elastic instability of the outstanding flange, causing wrinkling of the flange and twisting of the column at stresses less than those for general column instability as given by Equations (8–25) and (8–26). For outstanding flanges of cross sections such as I, H, +, and L, the flange instability stress can be estimated by

$$f_{cr} = 0.044E\frac{t^2}{b^2} \qquad (8\text{–}30)$$

where E is column modulus of elasticity, t thickness of the outstanding flange, and b width of the outstanding flange. If the joints between the column members are glued and reinforced with glued fillets, the instability stress increases to as much as 1.6 times that given by Equation (8–30).

Bending

Beams are subject to two kinds of instability: lateral–torsional buckling and progressive deflection under water ponding, both of which are determined by member stiffness.

Water Ponding

Roof beams that are insufficiently stiff or spaced too far apart for their given stiffness can fail by progressive deflection under the weight of water from steady rain or another continuous source. The critical beam spacing S_{cr} is given by

$$S_{cr} = \frac{m\pi^4 EI}{\rho L^4} \qquad (8\text{–}31)$$

where E is beam modulus of elasticity, I beam moment of inertia, ρ density of water (1,000 kg/m^3, 0.0361 lb/in^3), L beam length, and $m = 1$ for simple support or $m = 16/3$ for fixed-end condition. To prevent ponding, the beam spacing must be less than S_{cr}.

Lateral–Torsional Buckling

Since beams are compressed on the concave edge when bent under load, they can buckle by a combination of lateral deflection and twist. Because most wood beams are rectangular in cross section, the equations presented here are for rectangular members only. Beams of I, H, or other built-up cross section exhibit a more complex resistance to twisting and are more stable than the following equations would predict.

Long Beams—Long slender beams that are restrained against axial rotation at their points of support but are otherwise free to twist and to deflect laterally will buckle when the maximum bending stress f_b equals or exceeds the following critical value:

$$f_{b\,cr} = \frac{\pi^2 E_L}{\alpha^2} \qquad (8\text{–}32)$$

where α is the slenderness factor given by

$$\alpha = \sqrt{2\pi}\ \sqrt[4]{\frac{EI_y}{GK}}\ \frac{\sqrt{L_e h}}{b} \qquad (8\text{–}33)$$

where EI_y is lateral flexural rigidity equal to $E_L hb^3/12$, h is beam depth, b beam width, GK torsional rigidity defined in Equation (8–9), and L_e effective length determined by type of loading and support as given in Table 8–2. Equation (8–32) is valid for bending stresses below the proportional limit.

Short Beams—Short beams can buckle at stresses beyond the proportional limit. In view of the similarity of Equation (8–32) to Euler's formula (Eq. (8–25)) for column buckling, it is recommended that short-beam buckling be analyzed by using the column buckling criterion in Figure 8–7 applied with α in place of L/r on the abscissa

Table 8–2. Effective length for checking lateral–torsional stability of beams[a]

Support	Load	Effective length L_e
Simple support	Equal end moments	L
	Concentrated force at center	$\frac{0.742L}{1-2h/L}$
	Uniformly distributed force	$\frac{0.887L}{1-2h/L}$
Cantilever	Concentrated force at end	$\frac{0.783L}{1-2h/L}$
	Uniformly distributed force	$\frac{0.489L}{1-2h/L}$

[a]These values are conservative for beams with a width-to-depth ratio of less than 0.4. The load is assumed to act at the top edge of the beam.

and $f_{b\,cr}/F_b$ in place of f_{cr}/F_c on the ordinate. Here F_b is beam modulus of rupture.

Effect of Deck Support—The most common form of support against lateral deflection is a deck continuously attached to the top edge of the beam. If this deck is rigid against shear in the plane of the deck and is attached to the compression edge of the beam, the beam cannot buckle. In regions where the deck is attached to the tension edge of the beam, as where a beam is continuous over a support, the deck cannot be counted on to prevent buckling and restraint against axial rotation should be provided at the support point.

If the deck is not very rigid against in-plane shear, as for example standard 38-mm (nominal 2-in.) wood decking, Equation (8–32) and Figure 8–7 can still be used to check stability except that now the effective length is modified by dividing by θ, as given in Figure 8–8. The abscissa of this figure is a deck shear stiffness parameter τ given by

$$\tau = \frac{SG_D L^2}{EI_y} \tag{8–34}$$

where EI_y is lateral flexural rigidity as in Equation (8–33), S beam spacing, G_D in-plane shear rigidity of deck (ratio of shear force per unit length of edge to shear strain), and L actual beam length. This figure applies only to simply supported beams. Cantilevers with the deck on top have their tension edge supported and do not derive much support from the deck.

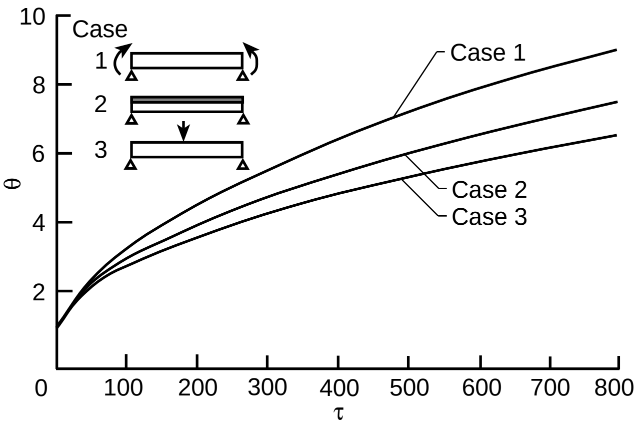

Figure 8–8. Increase in buckling stress resulting from attached deck; simply supported beams. To apply this graph, divide the effective length by θ.

Interaction of Buckling Modes

When two or more loads are acting and each of them has a critical value associated with a mode of buckling, the combination can produce buckling even though each load is less than its own critical value.

The general case of a beam of unbraced length l_e includes a primary (edgewise) moment M_1, a lateral (flatwise) moment M_2, and axial load P. The axial load creates a secondary moment on both edgewise and flatwise moments due to the deflection under combined loading given by Equation (8–7). In addition, the edgewise moment has an effect like the secondary moment effect on the flatwise moment.

The following equation contains two moment modification factors, one on the edgewise bending stress and one on the flatwise bending stress that includes the interaction of biaxial bending. The equation also contains a squared term for axial load to better predict experimental data:

$$\left(\frac{f_c}{F_c}\right)^2 + \frac{f_{b1} + 6(e_1/d_1)f_c(1.234 - 0.234\theta_{c1})}{\theta_{c1}F'_{b1}} + \frac{f_{b2} + 6(e_2/d_2)f_c(1.234 - 0.234\theta_{c2})}{\theta_{c2}F'_{b2}} \le 1.0 \tag{8–35}$$

where f is actual stress in compression, edgewise bending, or flatwise bending (subscripts c, b1, or b2, respectively), F buckling strength in compression or bending (a single prime denotes the strength is reduced for slenderness), e/d ratio of eccentricity of the axial compression to member depth ratio for edgewise or flatwise bending (subscripts 1 or 2, respectively), and θ_c moment magnification factors for edgewise and flatwise bending, given by

$$\theta_{c1} = 1 - \left(\frac{f_c}{F''_{c1}} + \frac{S}{S_{cr}} \right) \quad (8\text{–}36)$$

$$\theta_{c2} = 1 - \left(\frac{f_c}{F''_{c2}} + \frac{f_{b1} + 6(e_1/d_1) f_c}{F''_{b1}} \right) \quad (8\text{–}37)$$

$$F''_{c1} = \frac{0.822E}{(l_{e1}/d_1)^2} \quad (8\text{–}38)$$

$$F''_{c2} = \frac{0.822E}{(l_{e2}/d_2)^2} \quad (8\text{–}39)$$

$$F''_{b1} = \frac{1.44E}{l_e} \frac{d_2}{d_1} \quad (8\text{–}40)$$

where l_e is effective length of member and S and S_{cr} are previously defined ponding beam spacing.

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Chapter 9

Adhesive Bonding of Wood Materials

Charles B. Vick

Contents

Adhesive bonding of wood components has played an essential role in the development and growth of the forest products industry and has been a key factor in the efficient utilization of our timber resource. The largest use of adhesives is in the construction industry. By far, the largest amounts of adhesives are used to manufacture building materials, such as plywood, structural flakeboards, particleboards, fiberboards, structural framing and timbers, architectural doors, windows and frames, factory-laminated wood products, and glass fiber insulation. Adhesives are used in smaller amounts to assemble building materials in residential and industrial construction, particularly in panelized floor and wall systems. Significant amounts are also used in nonstructural applications, such as floor coverings, countertops, ceiling and wall tile, trim, and accessories.

Adhesives can effectively transfer and distribute stresses, thereby increasing the strength and stiffness of the composite. Effective transfer of stress from one member to another depends on the strength of the links in an imaginary chain of an adhesive-bonded joint. Thus, performance of the bonded joint depends on how well we understand and control the complexity of factors that constitute the individual links—wood, adhesive, and the interphasing regions between—which ultimately determine the strength of the chain.

Adhesion to Wood

The American Society for Testing and Materials (ASTM) defines an adhesive as a substance capable of holding materials together by surface attachment. An adherend is a substrate held to another substrate by an adhesive. Adhesion is the state in which two surfaces are held together by interfacial forces, which may be valence forces, interlocking action, or both. Valence forces are forces of attraction produced by the interactions of atoms, ions, and molecules that exist within and at the surfaces of both adhesive and adherend. Interlocking action, also called mechanical bonding, means surfaces are held together by an adhesive that has penetrated the porous surface while it is liquid, then anchored itself during solidification. The extent to which valence forces and interlocking action develop between adhesive polymers and wood adherends is uncertain, but both are generally acknowledged as essential for the most effective bonding. Bonding to porous surfaces, such as wood, paper, and textiles, was thought to be primarily mechanical, but now there is evidence supporting bonding by primary valence forces. In contrast, bonding to hard metal surfaces was believed to involve only valence forces, but this is no longer the accepted view. Metal surfaces roughened by chemical etching or made microscopically porous with a layer of oxide are capable of mechanical interlocking with an adhesive to produce exceptionally strong and durable bonds.

Mechanical interlocking is probably the primary mechanism by which adhesives adhere to porous structures, such as wood. Effective mechanical interlocking takes place when adhesives penetrate beyond the surface debris and damaged fibers into sound wood two to six cells deep. Deeper penetration into the fine microstructure increases the surface area of contact between adhesive and wood for more effective mechanical interlocking. The most durable structural bonds to wood are believed to develop not only when an adhesive penetrates deeply into cell cavities, but also when an adhesive diffuses into cell walls to make molecular-level contact with the hemicellulosics and cellulosics of wood. If an adhesive penetrates deeply enough into sound wood and becomes rigid enough upon curing, the strength of the bond can be expected to exceed the strength of the wood.

Physical forces of attraction composed of three intermolecular attraction forces are believed to be important to the formation of bonds between adhesive polymers and molecular structures of wood. Generally called van der Waal's forces, these include dipole–dipole forces, which are positively and negatively charged polar molecules that have strong attractions for other polar molecules; London forces, which include the weaker forces of attraction that nonpolar molecules have for each other; and hydrogen bonding, a special type of dipole–dipole force that accounts for strong attractions between positively charged hydrogen atoms of one polar molecule and the electronegative atom of another molecule. Hydrogen bonding forces are important in the interfacial attraction of polar adhesive polymers for the hemicellulosics and cellulosics, which are rich with polar hydroxyl groups. These physical forces of attraction, sometimes referred to as specific adhesion, are particularly important in wetting of water carriers and adsorption of adhesive polymers onto the molecular structures of wood.

Covalent chemical bonds form when atoms of nonmetals interact by sharing electrons to form molecules. The simplest example of a purely covalent bond is the sharing of electrons by two hydrogen atoms to form hydrogen. These covalent bonds are the strongest of chemical bonds; they are more than 11 times the strength of the hydrogen bond. Even though covalent chemical bonds between adhesive polymer and the molecular structure of wood seem a possibility, there is no clear evidence that such bonds constitute an important mechanism in adhesive bonding to wood.

For two wood adherends to be held together with maximum strength, a liquid adhesive must wet and spread freely to make intimate contact with both surfaces. Molecules of the adhesive must diffuse over and into each surface to make contact with the molecular structure of wood so that intermolecular forces of attraction between adhesive and wood can become effective. As will be discussed later, wood adherends, as well as other materials, differ widely in their attractive energies, bulk properties, surface roughness, and surface chemistry. Wood surfaces may appear to be smooth and flat, but on microscopic examination, they become peaks, valleys, and crevices, littered with loose fibers and other debris. Such surface conditions cause gas pockets and blockages that prevent complete wetting by the adhesive and introduce stress concentrations when the adhesive has cured. Thus, the liquid adhesive must have high wettability, coupled with a viscosity that will produce good capillary flow to penetrate sound wood structure, while displacing and absorbing air, water, and contaminants at the surface. Pressure is normally used to enhance wetting by forcing liquid adhesive to flow over the surfaces, displace air blockages, and penetrate to sound wood.

Wetting of a surface occurs when the contact angle (the angle between the edge of a drop of adhesive and the surface of wood) approaches zero. The contact angle approaches zero when the surface has high attractive energy, the adhesive has an affinity for the adherend, and the surface tension of the adhesive is low. If a drop of adhesive spreads to a thin film approaching zero contact angle, the adhesive has spread well and made intimate contact with the surface. The differences in wettabilities of various wood surfaces are illustrated by a simple water drop test in Figure 9–1.

The process of adhesion is essentially completed after transition of the adhesive from liquid to solid form. After the viscosity of a liquid adhesive has increased and the adhesive has solidified to the point where the film effectively resists shear and tensile forces tending to separate the surfaces, the surfaces are effectively bonded. An adhesive film changes from liquid to solid form by one of three mechanisms, although two may be involved in some curing mechanisms. This transition can be a physical change as in thermoplastic adhesives or it can be a chemical change as in thermosetting adhesives. In thermoplastics, the physical change to solid form may occur by either (a) loss of solvent from the adhesive through evaporation and diffusion into the wood, or (b) cooling of molten adhesive on a cooler surface. In thermosets, the solid form occurs through chemical polymerization into cross-linked structures that resist softening on heating. Most thermosetting wood adhesives contain water as a carrier; therefore, water also must be evaporated and absorbed by the wood so that the adhesive can cure completely.

Surface Properties of Wood Adherends

Because adhesives bond by surface attachment, the physical and chemical conditions of the adherend's surface is extremely important to satisfactory joint performance. Wood surfaces should be smooth, flat, and free of machine marks and other surface irregularities, including planer skips and crushed, torn, and chipped grain. The surface should be free of burnishes, exudates, oils, dirt, and other debris.

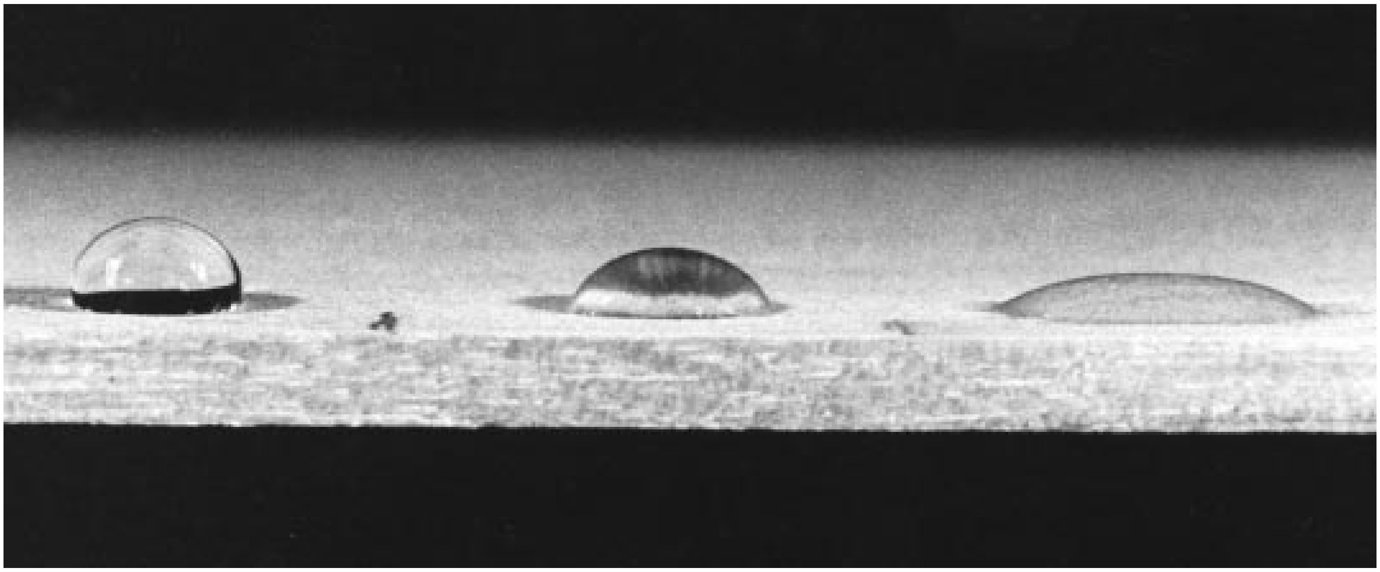

Figure 9–1. A simple water drop test shows differences in the wettability of a yellow birch veneer surface. Three drops were applied to the surface at the same time, then photographed after 30 s. The drop on the left retains a large contact angle on the aged, unsanded surface; the drop in the center has a smaller contact angle and improved wettability after the surface is renewed by two passes with 320-grit sandpaper; the drop on the right shows a small contact angle and good wettability after four passes with the sandpaper.

Overdrying and overheating deteriorates the physical condition of the wood surfaces by forcing extractives to diffuse to the surface, by reorienting surface molecules, and by irreversibly closing the larger micropores of cell walls. Wood surfaces can be chemically inactivated with respect to adhesion by airborne chemical contaminants, hydrophobic and chemically active extractives from the wood, oxidation and pyrolysis of wood bonding sites from overdrying, and impregnation with preservatives, fire retardants, and other chemicals. Unfortunately, some of these surface conditions are difficult to detect. Physical deterioration and chemical contamination interfere with essential wetting, flow, and penetration of adhesive but can also interfere with the cure and resulting cohesive strength of the adhesive.

Extractives on Surfaces

Extensive research indicates that extractives on wood surfaces are the principal physical and chemical contributors to surface inactivation, hence to poor wettability by adhesives. This is particularly true for resinous species, such as the southern pines and Douglas-fir. When subjected to high temperatures during processing, extractives diffuse to the surface where they concentrate and physically block adhesive contact with wood. Furthermore, resinous and oily exudates are hydrophobic; that is, they repel water. Most wood adhesives contain water as a carrier; therefore, they do not properly wet, flow, and penetrate extractive-covered surfaces. The acidity of extractives of some Southeast Asian hardwoods and oak species can interfere with the chemical cure of adhesives. The acid may accelerate the cure of an alkaline phenolic adhesive, causing the adhesive to gel prematurely and reducing its ability to wet, flow, and penetrate. In contrast, normal polymerization of an acidic adhesive, such as urea-formaldehyde, can be retarded by an alkaline wood surface, which would compromise the integrity of the adhesive film and bond.

A simple water test can reveal much about the state of inactivation of a wood surface and how difficult it may be to wet and bond with adhesive. As a first test, place a small drop of water on the surface and observe how it spreads and absorbs. If the drop remains a bead and does not begin to spread within 30 s, the surface is resistant to adhesive wetting (Fig. 9–1). Another water drop test can be used to estimate the degree of surface inactivation of veneer. Place a drop of water in an area on the earlywood of a flat-grain surface that does not have checks or splits in the area of the drop. Good wettability is indicated if the drop is absorbed within 20 min. If the drop has spread out but some water still remains on the surface after 40 min, then bonding problems are likely to occur. If after 40 min the water drop still retains much of its original shape with little spreading, then bonding problems from surface inactivation is a certainty.

Knife- and Abrasive-Planed Surfaces

Wood should be surfaced or resurfaced within 24 h before bonding to remove extractives and other physical and chemical contaminants that interfere with bonding. Surfacing also removes any unevenness that may have occurred from changes in moisture content. Parallel and flat surfaces allow the adhesive to flow freely and form a uniformly thin layer of adhesive that is essential to the best performance of water-based wood adhesives.

Experience and testing have proven that a smooth, knife-cut surface is best for bonding. Surfaces made by saws usually are rougher than those made by planers and jointers. However, surfaces sawn with special blades on properly set straight-line ripsaws are satisfactory for both structural and nonstructural joints. Precision sawing of wood joints rather than two-step sawing and jointing is commonplace in furniture manufacture for purposes of reducing costs for labor, equipment, and material. Unless the saws and feed works are well maintained, however, joints made with sawed surfaces will be weaker and less uniform in strength than those made with sharp planer or jointer knives. Dull cutting edges of planer or jointer knives crush and burnish the wood surface. The crushed and burnished surface inhibits adhesive wetting and penetration. If the adhesive does not completely penetrate crushed cells to restore their original strength, a weak joint results. Another simple water test can be used to detect a surface that has been damaged during machining. Wipe a very wet rag over a portion of the surface. After waiting for a minute, remove any remaining water by blotting with a paper towel. Then compare the roughness of the wet and dry surfaces. If the wetted area is much rougher than the dry area, then the surface has been damaged in machining. This damage will significantly reduce the strength of adhesive-bonded joints.

Abrasive planing with grit sizes from 24 to 80 causes surface and subsurface crushing of wood cells. Figure 9–2 shows cross sections of bondlines between undamaged, knife-planed Douglas-fir lumber compared with surfaces damaged by abrasive planing. Such damaged surfaces are inherently weak and result in poor bond strength. Similar damage can be caused by dull planer knives or saws. There is some evidence that sanding with grits finer than 100 may improve an abrasive-planed surface. However, abrasive-planing is not recommended for structural joints that will be subjected to high swelling and shrinkage stresses from water soaking and drying. If abrasive-planing is to be used before bonding, then belts must be kept clean and sharp, and sanding dust must be removed completely from the sanded surface.

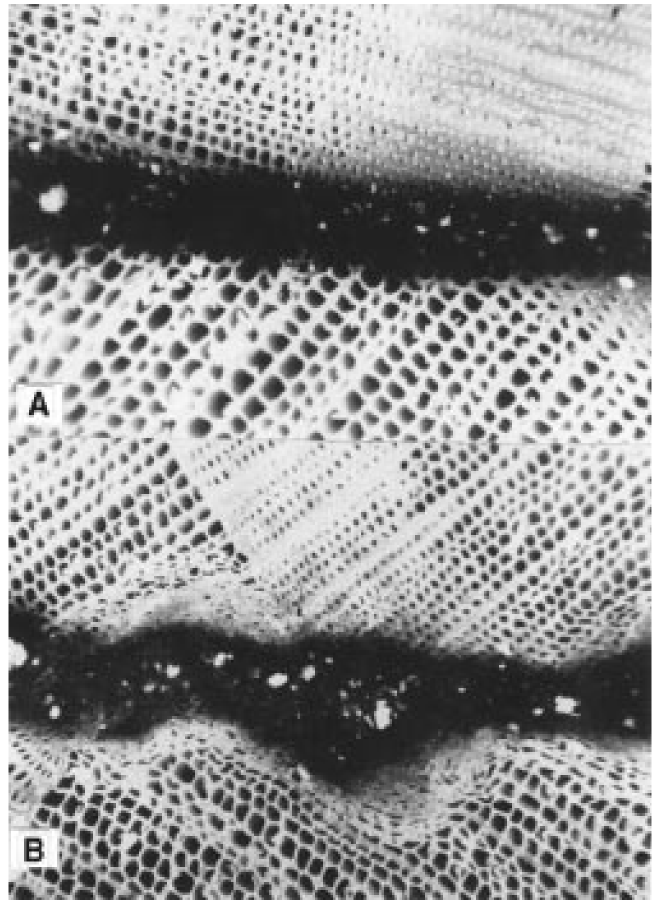

Figure 9–2. (A) Cross section of a bonded joint between two undamaged Douglas-fir surfaces that were planed with a sharp knife (120X). The wood cells are open, and their walls are distinct. The dark area at the center of micrograph is the adhesive bondline. (B) Cross section of a bonded joint between two damaged Douglas-fir surfaces abrasively planed with 36-grit sandpaper. The cells in and adjacent to the bondline are crushed, and their walls are indistinct.

Veneer Surfaces

The wood properties of veneer are essentially no different from those of lumber; however, manufacturing processes, including cutting, drying, and laminating into plywood, can drastically change physical and chemical surface properties of veneer. Special knowledge and attention to these characteristics are required to ensure good wetting, flow, and penetration of adhesive.

Rotary cutting produces continuous sheets of flat-grain veneer by rotating a log by its ends against a knife. As the knife peels veneer from the log, the knife forces the veneer away from the log at a sharp angle, thereby breaking or checking the veneer on the knife side. The checked side is commonly called the loose side, and the opposite side without checking is called the tight side. When rotary-cut veneer is used for faces in plywood, the loose side should be bonded with the tight side presented to view. Otherwise, open checks in the faces produce imperfections in any finish that may be applied. Adhesive overpenetration into lathe checks usually is not a problem if the adhesive spread rate is adjusted correctly.

Sliced veneer is produced in long strips by moving a squared log, called a flitch, against a knife. As in rotary cutting, the veneer is forced by the knife away from the flitch at a sharp angle, causing fine checking of the veneer on the knife side. This checked surface will show imperfections in a finished surface, so the loose side should be bonded and the tight side finished. For book-matched face veneers, where grain patterns of adjacent veneers are near mirror images, half the veneers will be loosely cut and must be finished, so the veneer must be cut as tightly as possible. Generally, hardwood face veneers are sliced to reveal the most attractive grain patterns.

Sawn veneer is produced in long narrow strips from flitches that have been selected and sawn for attractive grain patterns. The two sides of sawn veneer are free from knife checks, so either surface may be bonded or exposed to view with satisfactory results.

Veneer is dried promptly after cutting, using continuous, high temperature dryers that are heated either with steam or hot gases from wood-residue or gas-fired burners. Drying temperatures range from 170°C to 230°C (330°F to 446°F) for short periods. Drying veneer to very low moisture content levels at very high temperatures and drying at moderate temperatures for prolonged periods inactivates surfaces, causing poor wetting of veneer, hence poor bonding of the plywood. Residues deposited on veneer surfaces from incomplete combustion of gases and fuel oils can cause serious adhesion problems in plywood production.

Veneer selected for its attractive appearance, or for use in sanded grades of plywood, should be uniform in thickness, smooth, flat, free from deep checks, knots, holes, and decay, and have face grain suitable for the intended face grade. For plywood of the lower grades, defect requirements are not as restricted. For example, loosely cut veneer with frequent deep checks and large defects is suitable for structural plywood, but more adhesive is required than for tightly cut veneer. Higher spread rates compensate for overpenetration of adhesive into loosely cut veneer. When rotary-cut veneer is bonded into plywood, the tight side is usually bonded to the loose side, except that in one bondline, the loose side must be bonded to the loose side. This orientation permits the face veneer to be presented with its tight side facing outward for sanding and appearance.

Surfaces of Wood and Nonwood Composite Products

The surfaces of wood products such as plywood, structural flakeboard, particleboard, fiberboard, and hardboard generally have poor wettability relative to that of freshly cut, polar wood surfaces. Surfaces of these materials may have a glazed appearance indicating they have been inactivated by pressing at high temperatures. During hot pressing, resinous extractives migrate to the surface, adhesives on the outer surfaces of particles and fibers cure, and caul release agents remain on product surfaces—all of which inactivate or block surfaces from being wetted by water-based wood adhesives. Furthermore, the strength of bonds to the surfaces of these products is limited by the strength with which surface flakes, particles, and fibers are bound to the inner flakes, particles, and fibers of the product. A much lower bond strength can be expected to the surfaces of products of particulate structure than to products of natural wood structure. Adhesion to composite panel products having poor wettability (Fig. 9–1) can be improved by lightly sanding with 320-grit sandpaper. However, too much sanding can change a flat surface to an uneven surface and perhaps produce too much loose-fiber debris that would interfere with adhesion.

Metal foils and plastic films are commonly laminated to wood panels usually by product manufacturers. Although high cohesive strength is not required of adhesives to support these materials in an indoor environment, adhesives still must be reasonably compatible with both the wood and nonwood surfaces. If a bond of greater structural integrity is required to bond wood to heavier, rigid metals and plastics, then only epoxy, polyurethane, and other isocyanate-based adhesives may be sufficiently compatible with metals and plastics. Even then, cleaning or special preparation of the nonwood surfaces may be required to remove contaminants and chemically activate the surfaces. Composite materials are becoming more common as manufacturers learn to bond dissimilar materials to gain extraordinary composite properties or cost advantages not available from a single component. Composite materials in which nonpolar thermoplastics are successfully bonded to polar wood materials with the aid of coupling agents are becoming commonplace.

Metals are stronger and stiffer than wood and if bonded well enough to effectively transfer stresses between metal and wood, the mechanical properties of wood can be enhanced by the metal so that the resultant composite performs as a single material. Metal has a much higher energy surface than does wood. On exposure to air, oxides of the metal quickly form, and with moisture, gases, and debris adsorbed from the air, the surfaces quickly develop a low energy, weak boundary layer. To restore the high energy surfaces, a series of cleaning procedures are required to prepare the surfaces for structural bonding. Steps in surface preparation include cleaning with liquid or vapor organic solvents, abrading by sandblasting, alkaline washing, chemical etching, and priming with adhesive solutions or coupling agents.

Plastics are organic polymers that may be either thermoplastic (soften on heating) or thermosetting (cross-linked and resist softening on heating). Thermoplastics generally are not as strong and stiff as wood, but thermoset materials approximate and even exceed the mechanical properties of wood. When plastics contain fibrous reinforcing materials, such as fiberglass, strength and stiffness of the composite materials greatly exceed some of the mechanical properties of wood. In so doing, reinforced plastics that are effectively bonded to wood offer stronger and more cost-effective structural composites. The surfaces of plastics generally are low energy, nonpolar, and hydrophobic. Traditional aqueous-based wood adhesives are polar and hydrophilic, so they do not bond well to plastics. Epoxies, polyurethanes, and isocyanate-based adhesives are capable of bonding many plastics to wood. Adhesion to plastic surfaces occurs primarily by physical intermolecular attraction forces and, in some cases, hydrogen bonding. Abrading and chemical etching of plastic surfaces provide some mechanical interlocking, thereby increasing adhesion. Coupling agents are particularly useful for chemically bridging dissimilar materials. They have molecules that are of either unlike or like functionalities that are capable of reacting with both the adhesive and the surface of the adherend. Treatment of plastic surfaces with an inert gas, including oxygen plasma activated by radio-frequency energy, cleans and activates surfaces for enhanced adhesion. Grafting of monomers onto cleaned plastic surfaces by means of plasma polymerization creates a polar surface that is more compatible with adhesives.

Chemical treatment of wood with preservatives, fire retardants, and dimensional stabilizers interferes with adhesion to the treated wood. Types of chemical treatment and adhesives, conditions of joint assembly and adhesive cure, and prebonding chemical surface treatments have varied, interacting, and even strong effects on the strength and durability of bonds. Certain combinations of these factors can lead to excellent bonds, despite the interference from chemical treatments.

Lumber treated with chromated copper arsenate (CCA) preservatives dominates the treated wood market; however, very little of the CCA-treated wood is used in adhesively bonded lumber products. Commercial adhesives do not adhere to CCA-treated wood well enough to consistently meet rigorous industrial standards for resistance to delamination in accelerated exterior service tests. Analytical studies have shown that cellular surfaces of CCA-treated wood are thoroughly covered with microscopic-size deposits of mixtures of chromium, copper, and arsenic oxides that are physicochemically fixed to cell walls. The presence of these insoluble metallic deposits is so pervasive that intermolecular forces of attraction that normally act between polar wood and adhesive are physically blocked (Fig. 9–3). A new hydroxymethylated resorcinol (HMR) coupling agent greatly improves adhesion to CCA-treated wood when HMR is applied as a dilute aqueous primer on lumber surfaces before bonding. The HMR physicochemically couples phenol-resorcinol, epoxy, emulsion polymer–isocyanate, polymeric methylene diphenyl disocyanate, and melamine-urea adhesives to treated wood so that bonds can meet rigorous industrial standards for strength and durability.

Wood preservatives other than CCA, even nonacidic waterborne preservatives including emulsion types, interfere with adhesion of hot-pressed phenolic plywood adhesives, particularly as levels of chemical retention in the wood increase. Generally, preservatives containing boron, copper, and zinc interfere with the cure of phenolic resins, although assembly conditions can be optimized to improve bonding. Certain alkyl ammonium and fluoride-based salt preservatives have demonstrated limited interference with adhesion.

The most common fire-retarding chemicals used for wood are inorganic salts based on phosphorous, nitrogen, and boron. These acid salts release acid at elevated temperatures to decrease flammable volatiles and increase char in wood, thereby effectively reducing flame spread. A few salts release acid at temperatures lower than fire conditions, and in the presence of elevated temperature and moisture service conditions, increasing acidity leads to destructive hydrolysis of the wood. The acidity of the fire-retardant-treated wood, particularly at the elevated temperature and moisture conditions of hot-press curing, also inhibits the cure and bond formation of alkaline phenolic adhesive. By priming treated-wood surfaces with certain alkaline aqueous solutions before bonding and selecting resins of appropriate molecular-size distribution, strong and durable bonds can be made to certain fire-retardant-treated woods.

Acetylation is a chemical modification of wood that drastically reduces moisture-related dimensional changes and rate of biodeterioration. Acetic anhydride is reacted with the hydroxyl groups of hemicelluloses and lignin of wood. For every acetyl group reacted, one hydroxyl group is blocked from hydrogen bonding with a water molecule, and the result is lower affinity of acetylated wood for water. Reduced wettability from fewer available hydroxyl groups means poorer adhesion of aqueous-based wood adhesives. Adhesion is reduced to varying degrees among thermoplastic and thermosetting adhesives in proportion to their compatibility with the amount of nonpolar, hydrophobic acetate groups formed in the acetylated wood. Only room-temperature-curing resorcinolic adhesives and an acid-catalyzed phenolic hot-press adhesive have been found to develop durable bonds to acetylated wood. All other wood adhesives develop poorer bonds to acetylated wood than to untreated wood.

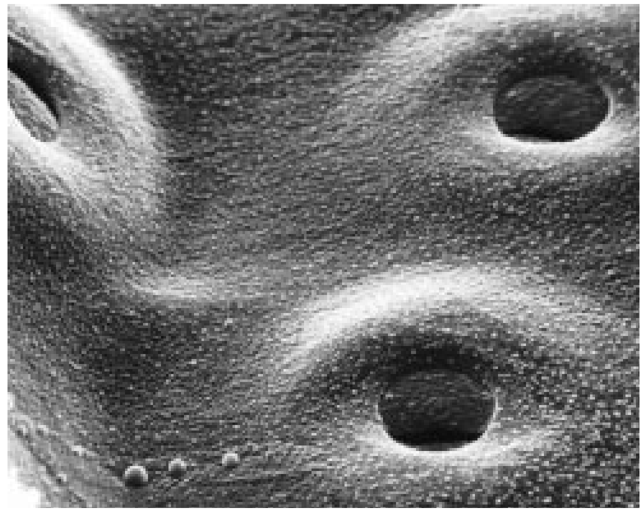

Figure 9–3. Surface of cell lumen of CCA-treated Southern Pine covered with chemically fixed deposits of insoluble mixture of chromium, copper, and arsenic oxides.

Physical Properties of Wood Adherends

Density and Porosity

The bondability of wood is not only affected by the surface properties of wood adherends but also by wood's physical properties, particularly density, porosity, moisture content, and dimensional movement.

Wood substance without void volume has a density approximating 1.5 g/cm^3 (93.6 lb/ft^3), regardless of the wood species. But density varies greatly between wood species, and even within a species, because species vary in void volume and thickness of cell walls. High density woods have thick walls and small lumen volumes, whereas low density woods have thin walls with large lumen volumes. The strength of wood is directly related to its density because thick-walled cells are capable of withstanding much greater stress than are thin-walled cells. Wood cells are an integral part of the wood–adhesive interphasing region; therefore, the adhesive bond must be at least as strong as the wood if the strength capability of the wood adherend is to be fully utilized.

The strength of adhesive bonds to wood increases with wood density up to a range of 0.7 to 0.8 g/cm^3 (43.7 to 49.9 lb/ft^3) (moisture content 12%). Above this level, joint strength decreases. Although strength increases with wood density, wood failure decreases gradually up to a density range of 0.7 to 0.8 g/cm^3 (43.7 to 49.9 lb/ft^3), then decreases more rapidly above 0.8 g/cm^3 (49.9 lb/ft^3). As wood density increases, high strength joints with high wood failure are more difficult to achieve consistently. (Wood failure means rupture of wood fibers during strength tests of adhesive bonds to wood. It is usually expressed as a percentage of the total bonded area of the joint.)

High density woods are difficult to bond for several reasons. Because of thicker cell walls and less lumen volume, adhesives do not penetrate easily, so important mechanical interlocking of adhesives is limited to one or two cells deep. Much greater pressure is required to compress stronger, stiffer, high density wood to bring contact between wood surface and adhesive. Higher concentrations of extractives that may interfere with the cure of adhesives are common in high density species, particularly domestic oaks and imported tropical hardwoods. The severe stresses produced by high density species as they change dimensions with changes in moisture content also contribute heavily to bonding difficulties.

Density is perhaps a crude indicator, but as previously noted, it is useful for estimating the bondability of a great variety of wood species. Table 9–1 categorizes commonly used domestic and imported species according to their relative ease of bonding. The categories for domestic woods are based on the average strength of side-grain joints of lumber as determined in laboratory tests and industrial experience. The laboratory tests included animal, casein, starch, urea-formaldehyde, and resorcinol-formaldehyde adhesives. The categories for imported woods are based on information found in the literature on bond strength, species properties, extractives content, and industrial experience. In most cases, the amount of data available for categorizing imported woods is not equivalent to that for domestic woods. However, a species that bonds poorly with one adhesive may develop much better bonds with another adhesive. A similar type of adhesive but with somewhat different working, penetration, curing, and even strength properties can often dramatically improve bondability of a given species. Adhesive suppliers will quite often adjust adhesive formulations to solve specific adhesion problems.

The void volume of wood, which can range from 46% to 80% of total volume, strongly affects the depth and direction that an adhesive flows. To attain the highest joint strength, the adhesive must penetrate and mechanically interlock several cells deep into sound, undamaged cell structure. In wood, porosity varies according to the grain direction. It is most porous on end-grain surfaces, being many times greater than on radial or tangential surfaces. Adhesives penetrate deeply into open fibers and vessels along the grain, so deeply that overpenetration occurs when pressure is applied to end-grain surfaces. This is a primary reason why it is so difficult to form strong, load-bearing bondlines in butt joints. Across the grain, porosity is limited because of fewer pathways in which adhesive can flow, so overpenetration under pressure generally is not a problem with a properly formulated adhesive.

The porosity of hardwoods and softwoods, both as species groups and as species within a group, varies greatly, which dramatically affects the amount and direction of adhesive flow. Highly porous softwoods, such as the southern pines, have fiber lumens that are interconnected by open pits. Pits are the small openings between fibers that permit lateral transfer of fluids in living trees. They form a complex capillary system that also allows adhesives to penetrate deeply, even in tangential and radial directions. The relatively large vessels in hardwoods have no end walls, so adhesive can penetrate indefinitely along the end grain. The remaining fibers have relatively few pits for lateral transfer of adhesive, except that hardwoods, such as the red oaks, have radially oriented rays that can allow excessive flow and overpenetration. Although adhesives for hardwoods and softwoods generally differ by chemical type according to product markets, adhesives must be specifically formulated for hardwoods or softwoods, including specific species within the groups, or have adjustable working properties for specific manufacturing situations.

Moisture Content and Dimensional Changes

Water occurs naturally in living trees—as free water in cell lumens and as adsorbed water within cell walls. Total water content of wood can range well above 200% (based on oven-dry weight), but when the free water is removed from cell lumens by drying, approximately 30% remains bound within cell walls. Water has strong molecular attraction to wood, primarily through hydrogen bonding with hydroxyl groups of wood cellulosics. Therefore, cell walls remain saturated with moisture (called the fiber saturation point) until the moisture content of the surrounding air falls below that of saturated cell walls. Actual moisture content at fiber saturation (roughly 30%) varies, depending on species, tree, temperature, and pressure. This is the critical point at which wood begins to shrink. If wood has dried below the fiber saturation point, then regains moisture, the wood will swell. These dimensional changes differ with the three principal directions, or grain directions in wood, that is, longitudinal, radial, and tangential, with intermediate changes varying with the angle between the principal directions. Longitudinal dimensional change along the grain is least and amounts to less than 1% in drying from fiber saturation point to oven-dry. Dimensional change is greatest across the grain, but the

Table 9–1. Categories of selected wood species according to ease of bonding

U.S. hardwoods	U.S. softwoods	Imported woods	
		Bond easily[a]	
Alder	Fir	Balsa	Hura
Aspen	White	Cativo	Purpleheart
Basswood	Grand	Courbaril	Roble
Cottonwood	Noble	Determa[b]	
Chestnut, American	Pacific		
Magnolia	Pine		
Willow, black	Eastern white		
	Western white		
	Redcedar, western		
	Redwood		
	Spruce, Sitka		
		Bond well[c]	
Butternut	Douglas-fir	Afromosia	Meranti (lauan)
Elm	Larch, western[d]	Andiroba	Light red
American	Pine	Angelique	White
Rock	Sugar	Avodire	Yellow
Hackberry	Ponderosa	Banak	Obeche
Maple, soft	Redcedar, eastern	Iroko	Okoume
Sweetgum		Jarrah	Opepe
Sycamore		Limba	Peroba rosa
Tupelo		Mahogany	Sapele
Walnut, black		African	Spanish-cedar
Yellow-poplar		American	Sucupira
			Wallaba
		Bond satisfactorily[e]	
Ash, white	Yellow-cedar	Angelin	Meranti (lauan), dark red
Beech, American	Port-Orford-cedar	Azobe	Pau marfim
Birch	Pines, southern	Benge	Parana-pine
Sweet		Bubinga	Pine
Yellow		Karri	Caribbean
Cherry			Radiata
Hickory			Ramin
Pecan			
True			
Madrone			
Maple, hard			
Oak			
Red[b]			
White[b]			
		Bond with difficulty[f]	
Osage-orange		Balata	Keruing
Persimmon		Balau	Lapacho
		Greenheart	Lignumvitae
		Kaneelhart	Rosewood
		Kapur	Teak

[a]Bond very easily with adhesives of a wide range of properties and under a wide range of bonding conditions.
[b]Difficult to bond with phenol-formaldehyde adhesive.
[c]Bond well with a fairly wide range of adhesives under a moderately wide range of bonding conditions.
[d]Wood from butt logs with high extractive content is difficult to bond.
[e]Bond satisfactorily with good-quality adhesives under well-controlled bonding conditions.
[f]Satisfactory results require careful selection of adhesives and very close control of bonding conditions; may require special surface treatment.

amounts differ with the direction; dimensional change varies with and within species. As a rule of thumb, tangential dimensional change is about twice that of the radial direction; but again, there are variations by species. (See Ch. 3 for a detailed discussion of wood moisture relations.)

Dimensional changes that accompany changes in moisture content have broad-ranging and significant consequences on performance of bonded joints. As wood in bonded assemblies swells and shrinks, stresses develop that can be great enough to rupture adhesive bond and wood. Ruptures may develop when adjacent pieces of wood in a bonded joint differ in grain direction and shrinkage coefficients, for example, radial grain bonded to tangential grain, or in the worst case, longitudinal grain bonded to either tangential or radial grain. Even if moisture content levels in adjacent pieces are equal, but changing, stresses could be severe. Moreover, if moisture content in one piece is at equilibrium with surrounding air, that is, stable, but the other piece with differing grain direction is shrinking as it approaches equilibrium moisture content (EMC), then resultant stresses would be compounded and almost sure to rupture either the adhesive bond or the wood, whichever is weaker. Some wood adhesives are elastic enough to yield to stresses so that fracture does not occur. Structural wood adhesives have greater moduli of elasticity than wood and can effectively transfer stresses from one adherend to the other without failure. However, if stresses are great enough from extraordinary moisture content changes within adjacent pieces of wood of differing shrinkage coefficients, then fracture in either wood or a poor bond is almost inevitable. Severe stresses on bondlines can be minimized by bonding pieces of wood with compatible grain directions of low shrinkage coefficients at a uniform moisture content equivalent to that which the bonded assembly will encounter in service.

The amount of moisture in wood combined with water in adhesive will greatly influence the wetting, flow, penetration, and even cure of aqueous wood adhesives. In general, these adhesives bond satisfactorily across moisture content levels ranging from 6% to 14% and even below and above this range when adhesives are formulated for specialized processing. The optimum moisture content range for bonding a specific product with a specific adhesive is determined from practical experience and product performance. Aqueous adhesives tend to dry out when applied to wood below 6% moisture content. Wood absorbs water from the adhesive so quickly that adhesive flow and penetration into the wood is drastically inhibited, even under high pressure. Wood may become so dry below 3% moisture content that it temporarily resists wetting by the adhesive because insufficient water remains bound to the wood to establish intermolecular attraction forces with water in the adhesive.

When wood contains excess amounts of moisture, then less water and adhesive can be absorbed by the wood. This leads to excessive adhesive mobility, followed by squeeze-out when pressure is applied. Control of moisture content is particularly critical to bonding in hot presses because excess moisture increases adhesive mobility, followed by overpenetration of the adhesive. Furthermore, high vapor pressure builds internally as water boils, and on release of platen pressure, sudden release of internal pressure actually separates laminates along the bondlines, called blows. Even if blows do not occur, excess moisture within thermosetting adhesives can prevent complete cross-linking with accompanying weakened adhesive film and bond. Appropriate moisture content levels of wood for bonding by hot-press methods are well known, as are target moisture content levels for satisfactory service of wood products throughout the United States. However, control of moisture content in bonding wood materials is not easily achieved. This is discussed in the Moisture Content Control section.

Adhesives

Composition

Organic polymers of either natural or synthetic origin are the major chemical ingredients in all formulations of wood adhesives. According to ASTM, a polymer is a compound formed by the reaction of simple molecules having functional groups that permit their combination to proceed to higher molecular weights under suitable conditions. Polysaccharides and proteins are high molecular weight natural polymers derived from plants and animals. Animal, blood, hide, casein, starch, soybean, dextrin, and cellulosic adhesives are all derived from the natural polymers found in these indicated sources. They have been used as adhesives for centuries and are still in use today, although they have been replaced mostly by adhesives made with synthetic polymers. The first wood adhesives based on synthetic polymers were produced commercially during the 1930s. This marked the beginning of fundamental changes in composition of adhesives from natural to synthesized polymers. These adhesives could not only be stronger, more rigid, and more durable than wood, but also have much greater resistance to water than adhesives from natural polymers.

Synthetic polymers are chemically designed and formulated into adhesives to perform a great variety of bonding functions. Whether the base polymer is thermoplastic or thermosetting has a major influence on how an adhesive will perform in service. Thermoplastics are long-chain polymers that soften and flow on heating, then harden again by cooling. They generally have less resistance to heat, moisture, and long-term static loading than do thermosetting polymers. Common wood adhesives that are based on thermoplastic polymers include polyvinyl acetate emulsions, elastomerics, contacts, and hot-melts. Thermosetting polymers make excellent structural adhesives because they undergo irreversible chemical change, and on reheating, they do not soften and flow again. They form cross-linked polymers that have high strength, have resistance to moisture and other chemicals, and are rigid enough to support high, long-term static loads without deforming. Phenolic, resorcinolic, melamine, isocyanate, urea, and epoxy are examples of types of wood adhesives that are based on thermosetting polymers.

A formulation of wood adhesive consists of a mixture of several chemically active and inert materials that vary in proportion with the basic adhesive polymer, which enhances performance, whether it be working characteristics, strength properties, shelf life, or durability. Solvents disperse or dissolve adhesive polymers, act as carriers of polymer and additives, aid wetting, and control flow and penetration of the adhesive. Water is used as the carrier for most wood adhesives, primarily because water readily absorbs into wood, is inexpensive, and is free of toxicity problems. Adhesive polymers can be brought into intimate, even molecular, contact with wood by water as the carrier. Organic solvents are used with elastomeric and contact adhesives, although water-based adhesive systems have lower toxicity and flammability. Fillers of both organic and inorganic origins contribute to rheological control of the fluid system, particularly in reducing the spreading and penetrating of the adhesive into wood. Reinforcing fibers, mostly inert and of organic origins, can enhance an adhesive film's mechanical properties, especially toughness, impact resistance, and shrinkage. Extenders are filler-like organic materials that may have sufficient chemical activity to improve adhesion to a small degree, but they are used primarily to control flow and other working characteristics, without excess sacrifice of adhesion capability, as is the case with most fillers.

Certain chemicals and polymeric systems plasticize adhesive polymers, and others are used to enhance tackiness. Plasticizers, such as dibutyl phthalate, are used to soften brittle vinyl acetate homopolymer in polyvinyl acetate emulsion adhesives, which facilitates diffusion of adhesive and formation of a flexible adhesive film from the emulsion at and below room temperature. Phenolic polymers are used as tackifiers and adhesion promoters in neoprene and nitrile rubber contact adhesives. Reactive polymeric fortifiers, such as melamine-formaldehyde, can be substituted in limited proportions in urea-formaldehyde adhesives to improve resistance to moisture and heat. Phenol-formaldehyde may be substituted for resorcinol-formaldehyde to reduce adhesive costs, without sacrificing adhesive strength and durability.

Catalysts are chemicals used to accelerate the rate of chemical reaction of polymeric components. Acids, bases, salts, peroxides, and sulfur compounds are a few examples of catalysts. Catalysts do not become a part of the reacted compound; they simply increase the rate of reaction. Hardeners are added to base polymers as reactive components, and they do become a part of the reacted compound. Examples are an amine hardener added to epoxy and formaldehyde added to resorcinol—all produce cross-linking reactions to solidify the adhesive. Other chemicals, such as antioxidants, acid scavengers, preservatives, wetting agents, defoamers, even colorants, may be added to control or eliminate some of the less desirable characteristics of certain adhesive formulations.

Health and Safety

Wood adhesives contain chemicals that are toxic to people if they are exposed to sufficient concentrations for prolonged periods. Generally, it is accepted that wood adhesives in a cured state do not present toxicity problems. A notable exception is urea-formaldehyde adhesive, which can release low concentrations of formaldehyde from bonded wood products under certain service conditions. Formaldehyde is a toxic gas that can react with proteins of the body to cause irritation and, in some cases, inflammation of membranes of eyes, nose, and throat. It is a suspected carcinogen, based on laboratory experiments with rats. Considerable research has led to new adhesive formulations with significantly reduced levels of formaldehyde emissions in both manufacturing operations and bonded wood products. Phenol-formaldehyde adhesives, which are used to manufacture plywood, flakeboard, and fiberglass insulation, also contain formaldehyde. However, formaldehyde is efficiently consumed in the curing reaction, and the highly durable phenol-formaldehyde, resorcinol-formaldehyde, and phenol-resorcinol-formaldehyde polymers do not chemically break down in service to release toxic gas.

Diisocyanates are highly reactive chemicals that polymerize rapidly on contact with strong alkali, mineral acids, and water. Polymeric methylene diphenyl diisocyanate (PMDI) adhesives develop strong and durable bonds to wood, so they are now widely used to manufacture composite wood products. They are potentially hazardous if mishandled, but the low vapor pressure of PMDI adhesives coupled with adequate ventilation to remove airborne PMDI on dust particles, permits manufacturing plants to operate safely. Properly cured PMDI adhesives are not considered hazardous in bonded wood products.

Construction and contact adhesives contain organic solvents that have low flash points. If these adhesives are used in unventilated areas where concentrations build to dangerously high levels, explosions can occur with an ignition source. Some adhesive producers now offer less flammable formulations based on chlorinated solvents. Organic solvents in these adhesives are toxic, but by following the manufacturer's handling and use instructions, coupled with adequate ventilation, harmful effects can be avoided.

Health and safety regulations require that toxic and hazardous chemicals be identified and visibly labeled to warn of their dangers. Material safety data sheets (MSDS) or instructions are provided with adhesive products to advise of proper handling procedures, protective gear and clothing, and procedures for dealing with spills and fire, as well as to offer guidance for first-aid and professional treatment of injuries. The statements made in this section concerning safety of adhesives and effects on the health of the user are general and not meant to be all inclusive. The user should consult the MSDS and follow the manufacturer's instructions and precautions before using any adhesive.

Strength and Durability

The ability of an adhesive to transfer load from one member of an assembly to another and to maintain integrity of the assembly under the expected conditions of service will govern the choice of adhesive for a given application.

In building construction, adhesives that contribute strength and stiffness during the life of the structure are considered structural. They generally are stronger and stiffer than the wood members. Structural bonds are critical because bond failure could result in serious damage to the structure, even loss of life. Examples of structural applications include glued-laminated beams, prefabricated I-joists, and stressed-skin panels. Adhesives that are strongest, most rigid, and most resistant to deterioration in service, unfortunately, are those least tolerant of wide variations in wood surface condition, wood moisture content, and assembly conditions including pressures, temperatures, and curing conditions. Examples of rigid structural adhesives include phenolic, resorcinol, melamine, urea, and casein (Table 9–2).

Adhesives are further categorized in Table 9–2 as to how well they transfer load relative to wood as the service environment becomes more severe. Structural adhesives that maintain their strength and rigidity under the most severe cyclic water-saturation and drying are considered fully exterior adhesives. Rigid adhesives that lose their ability to transfer load faster than does wood as service conditions worsen, particularly with regard to moisture, are considered interior adhesives. Between exterior and interior adhesives are the intermediate adhesives that maintain strength and rigidity in short-term water soaking but deteriorate faster than wood during long-term exposure to water and heat. Adhesives that are the weakest, least rigid, and least resistant to severe service conditions are those most tolerant of wide variations in wood surface, assembly, and curing conditions.

Semistructural adhesives impart strength and stiffness to an adhesive-bonded assembly, and in some instances, they may be as strong and rigid as wood. However, semistructural adhesives generally do not withstand long-term static loading without deformation. They are capable of short-term exposure to water but not long-term saturation, hence their limited exterior classification. Examples are cross-linking polyvinyl acetate and polyurethane adhesives. Another example of the semistructural adhesive application is the nailed–glued assembly where failure of the bond would not cause serious loss of structural integrity because the load would be carried by mechanical fasteners.

Table 9–2. Wood adhesives categorized according to their expected structural performance at varying levels of environmental exposure [a,b]

Structural integrity	Service environment	Adhesive type
Structural	Fully exterior (withstands long-term water soaking and drying)	Phenol-formaldehyde Resorcinol-formaldehyde Phenol-resorcinol-formaldehyde Emulsion polymer/isocyanate Melamine-formaldehyde
	Limited exterior (withstands short-term water soaking)	Melamine-urea-formaldehyde Isocyanate Epoxy
	Interior (withstands short-term high humidity)	Urea-formaldehyde Casein
Semistructural	Limited exterior	Cross-linked polyvinyl acetate Polyurethane
Nonstructural	Interior	Polyvinyl acetate Animal Soybean Elastomeric construction Elastomeric contact Hot-melt Starch

[a]Assignment of an adhesive type to only one structural/service environment category does not exclude certain adhesive formulations from falling into the next higher or lower category.
[b]Priming wood surfaces with hydroxymethylated resorcinol coupling agent improves resistance to delamination of epoxy, isocyanate, emulsion polymer/isocyanate, melamine and urea, phenolic, and resorcinolic adhesives in exterior service environment, particularly bonds to CCA-treated lumber.

Nonstructural adhesives typically support the dead weight of the material being bonded and can equal the strength and rigidity of wood in the dry condition. However, on exposure to water or high humidity, nonstructural adhesives quickly lose their load transfer ability. Examples are adhesives used for bonding wall tiles and fixtures.

Elastomeric construction adhesives are categorized as nonstructural. However, they are used normally for field assembly of panelized floor and wall systems in the light-frame construction industry. Nails are used in the assembly so that if failure did occur in the adhesive bond, the structural load would be carried by nails. The adhesive enables the nailed assembly to act as a composite with increased stiffness. With nails providing structural safety in this application, elastomeric adhesives could be included in the semistructural category.

Some adhesives listed in Table 9–2 could be included easily in more than one category because they can be formulated for a broad range of applications. Isocyanate and polyurethane adhesives are examples. Polymeric methylene diphenyl diisocyanates of low molecular weight develop highly durable bonds in structural flakeboards, although flakeboard products deteriorate from swelling and shrinkage stresses. One-part polyurethane adhesives have highly durable adhesive films, but as molecular weight increases, adhesion to porous wood generally decreases and bonds become increasingly susceptible to deterioration from swelling and shrinkage stresses. Polyurethane adhesives liberate carbon dioxide on reaction with water. As a result, they foam, and in thick bondlines, polyurethane bonds become more deformable under static loading. Two-part polyurethanes can be formulated for rigidity, depending on the degree of structural loading required.

Adhesive Selection

Adhesive selection begins by considering the types of wood adhesives, along with their strength and durability, preparation and use characteristics, and typical applications, as shown in abbreviated form in Table 9–3. Their relative strength and durability are categorized into levels of structural integrity (Table 9–2) and were discussed previously. Adhesive selection for a wood product manufacturer may begin as a cooperative effort between the manufacturer and an adhesive supplier. Together, they completely review the product, its intended service environment, and all production processes and equipment before choosing an appropriate adhesive. Whatever the approach to adhesive selection might be, the following general discussion should be helpful.

A broad array of adhesive types, with significant variations within each type, are available for bonding wood materials, even for bonding wood to nonwood materials. The selection process begins with determining which adhesives are compatible with the physical and chemical properties of the adherends, particularly their surface properties. The polar, aqueous wood adhesive must be capable of wetting the usually polar wood surface, within its normal variations in hydrophilicity. As the adhesive wets, it must have flow properties that enable it to spread over surfaces of variable roughness and to penetrate wood structures that differ in porosity, with respect to grain orientation at the bondline. The adhesive must make molecular contact with the lignocellulosics of wood and penetrate deeply enough to mechanically interlock with the wood's cell structure. Metals and plastics cannot be penetrated, so these materials generally cannot be bonded with aqueous wood adhesives. However, nonaqueous, 100% solids adhesives, including epoxy, isocyanate, and polyurethane, are capable of sound bonds to nonwood and wood materials.

The structural integrity expected of the adhesive bond under anticipated service loads in the presence of expected environmental exposure conditions should be one of the foremost considerations. To intelligently select an adhesive for a given bonded assembly, it is necessary to have an approximation of the nature, direction, level, and duration of loading that the assembly and bondlines must withstand. Furthermore, it is essential to know the range and duration of temperature and moisture content levels to which bondlines will be subjected. For example, prolonged exposure to high moisture content levels will significantly reduce the load-carrying ability of any adhesive in a wood joint. Failure to give full consideration to these factors could risk structural failure of the bonded assembly, even severe personal injury.

There may be need for tradeoffs between bonding requirements of adhesives and their resistance to stress, duration of load, and service environment. Adhesives that are the strongest, most rigid, and durable are generally those least tolerant of bonding conditions, including wood moisture content, surface roughness, cleanliness, inactivation, grain orientation, bondline thickness, and pressure and temperature of cure. Adhesives that are the weakest, least rigid, and least resistant to service conditions are those most tolerant of bonding conditions. Many adhesives are positioned between these extremes of bonding requirements and performance (Tables 9–2 and 9–3).

When a group of adhesives with suitable performance capabilities for a particular bonded assembly has been determined, the user also must choose within that group an adhesive that can be mixed, applied, and cured with available equipment or consider the cost of purchasing equipment to meet specific working properties of another adhesive. Important working properties must be considered when making cost decisions. The working life of an adhesive is the time between mixing and the end of its useful life when it becomes too viscous to properly wet and flow over a surface.

If an adhesive requires mixing with a hardener or catalyst, then mixing and application equipment appropriate for the working life must be considered. Given the consistency of an adhesive, specific types of application equipment are required. Depending on the size of the spreading operation, the equipment can range from brush to roll-spreader to extruder to spray to meter-mixed extrusion. Wood adhesives, including phenolic, melamine, urea, and isocyanate adhesives,

Table 9–3. Working and strength properties of adhesives, with typical uses

Type	Form and color	Preparation and application	Strength properties	Typical uses
Natural origin				
Animal, protein	Solid and liquid; brown to white bondline	Solid form added to water, soaked, and melted; adhesive kept warm during application; liquid form applied directly; both pressed at room temperature; bonding process must be adjusted for small changes in temperature	High dry strength; low resistance to water and damp atmosphere	Assembly of furniture and stringed instruments; repairs of antique furniture
Blood, protein	Solid and partially dried whole blood; dark red to black bondline	Mixed with cold water, lime, caustic soda, and other chemicals; applied at room temperature; pressed either at room temperature or 120°C (250°F) and higher	High dry strength; moderate resistance to water and damp atmosphere and to microorganisms	Interior-type softwood plywood, some times in combination with soybean adhesive; mostly replaced by phenolic adhesive
Casein, protein	Powder with added chemicals; white to tan bondline	Mixed with water; applied and pressed at room temperature	High dry strength; moderate resistance to water, damp atmospheres, and intermediate temperatures; not suitable for exterior uses	Interior doors; discontinued use in laminated timbers
Soybean, protein	Powder with added chemicals; white to tan, similar color in bondline	Mixed with cold water, lime, caustic soda, and other chemicals; applied and pressed at room temperatures, but more frequently hot pressed when blended with blood adhesive	Moderate to low dry strength; moderate to low resistance to water and damp atmospheres; moderate resistance to intermediate temperatures	Softwood plywood for interior use, now replaced by phenolic adhesive. New fast-setting resorcinol-soybean adhesives for fingerjointing of lumber being developed
Lignocellulosic residues and extracts	Powder or liquid; may be blended with phenolic adhesive; dark brown bondline	Blended with extender and filler by user; adhesive cured in hot-press 130°C to 150°C (266°F to 300°F) similar to phenolic adhesive	Good dry strength; moderate to good wet strength; durability improved by blending with phenolic adhesive	Partial replacement for phenolic adhesive in composite and plywood panel products
Synthetic origin				
Cross-linkable polyvinyl acetate emulsion	Liquid, similar to polyvinyl acetate emulsions but includes copolymers capable of cross-linking with a separate catalyst; white to tan with colorless bondline	Liquid emulsion mixed with catalyst; cure at room temperature or at elevated temperature in hot press and radio-frequency press	High dry strength; improved resistance to moisture and elevated temperatures, particularly long-term performance in moist environment	Interior and exterior doors; moulding and architectural woodwork; cellulosic overlays
Elastomeric contact	Viscous liquid, typically neoprene or styrene-butadine elastomers in organic solvent or water emulsion; tan to yellow	Liquid applied directly to both surfaces, partially dried after spreading and before pressing; roller-pressing at room temperature produces instant bonding	Strength develops immediately upon pressing, increases slowly over a period of weeks; dry strengths much lower than those of conventional wood adhesives; low resistance to water and damp atmospheres; adhesive film readily yields under static load	On-the-job bonding of decorative tops to kitchen counters; factory lamination of wood, paper, metal, and plastic sheet materials
Elastomeric mastic (construction adhesive)	Putty like consistency, synthetic or natural elastomers in organic solvent or latex emulsions; tan, yellow, gray	Mastic extruded in bead to framing members by caulking gun or like pressure equipment; nailing required to hold materials in place during setting and service	Strength develops slowly over several weeks; dry strength lower than conventional wood adhesives; resistant to water and moist atmospheres; tolerant of out door assembly conditions; gap-filling; nailing required to ensure structural integrity	Lumber to plywood in floor and wall systems; laminating gypsum board and rigid foam insulating; assembly of panel system in manufactured homes
Emulsion polymer/isocyanate	Liquid emulsion and separate isocyanate hardener; white with hardener; colorless bondline	Emulsion and hardener mixed by user; reactive on mixing with controllable pot-life and curing time; cured at room and elevated temperatures; radio-frequency curable; high pressure required	High dry and wet strength; very resistant to water and damp atmosphere; very resistant to prolonged and repeated wetting and drying; adheres to metals and plastics	Laminated beams for interior and exterior use; lamination of plywood to steel metals and plastics; doors and architectural materials

Table 9–3. Working and strength properties of adhesives, with typical uses—con.

Type	Form and color	Preparation and application	Strength properties	Typical uses
Epoxy	Liquid resin and hardener supplied as two parts; completely reactive leaving no free solvent; clear to amber; colorless bondline	Resin and hardener mixed by user; reactive with limited pot-life; cured at room or elevated temperatures; only low pressure required for bond development	High dry and wet strength to wood, metal, glass, and plastic; formulations for wood resist water and damp atmospheres; delaminate with repeated wetting and drying; gap-filling	Laminating veneer and lumber in cold-molded wood boat hulls; assembly of wood components in aircraft; lamination of architectural railings and posts; repair of laminated wood beams and architectural building components; laminating sports equipment; general purpose home and shop
Hot melt	Solid blocks, pellets, ribbons, rods, or films; solvent-free; white to tan; near colorless bondline	Solid form melted for spreading; bond formed on solidification; requires special application equipment for controlling melt and flow	Develops strength quickly on cooling; lower strength than conventional wood adhesives; moderate resistance to moisture; gap-filling with minimal penetration	Edge-banding of panels; plastic lamination; patching; film and paper overlays; furniture assembly; general purpose home and shop
Isocyanate	Liquid containing isomers and oligomers of methylene diphenyl diisocyanate; light brown liquid and clear bondline	Adhesive applied directly by spray; reactive with water; requires high temperature and high pressure for best bond development in flake boards	High dry and wet strength; very resistant to water and damp atmosphere; adheres to metals and plastics	Flakeboards; strand-wood products
Melamine and melamine-urea	Powder with blended catalyst; may be blended up to 40% with urea; white to tan; colorless bondline	Mixed with water; cured in hot press at 120°C to 150°C (250°F to 300°F); particularly suited for fast curing in high-frequency presses	High dry and wet strength; very resistant to water and damp atmospheres	Melamine-urea primary adhesive for durable bonds in hardwood plywood; end-jointing and edge-gluing of lumber; and scarf joining softwood plywood
Phenolic	Liquid, powder, and dry film; dark red bondline	Liquid blended with extenders and fillers by user; film inserted directly between laminates; powder applied directly to flakes in composites; all formulations cured in hot press at 120°C to 150°C (250°F to 300°F) up to 200°C (392°F) in flakeboards	High dry and wet strength; very resistant to water and damp atmospheres; more resistant than wood to high temperatures and chemical aging	Primary adhesive for exterior softwood plywood, flakeboard, and hardboard
Polyvinyl acetate emulsion	Liquid ready to use; often polymerized with other polymers; white to tan to yellow; colorless bondline	Liquid applied directly; pressed at room temperatures and in high-frequency press	High dry strength; low resistance to moisture and elevated temperatures; joints yield under continued stress	Furniture; flush doors; plastic laminates; panelized floor and wall systems in manufactured housing; general purpose in home and shop
Polyurethane	Low viscosity liquid to high viscosity mastic; supplied as one part; two-part systems completely reactive; color varies from clear to brown; colorless bondline	Adhesive applied directly to one surface, preferably to water-misted surface; reactive with moisture on surface and in air; cures at room temperature; high pressure required, but mastic required only pressure from nailing	High dry and wet strength; resistant to water and damp atmosphere; limited resistance to prolonged and repeated wetting and drying; gap-filling	General purpose home and shop; construction adhesive for panelized floor and wall systems; laminating plywood to metal and plastic sheet materials; specialty laminates; installation of gypsum board
Resorcinol and phenol-resorcinol	Liquid resin and powdered hardener supplied as two parts; phenol may be copolymerized with resorcinol; dark red bondline	Liquid mixed with powdered or liquid hardener; resorcinol adhesives cure at room temperatures; phenol-resorcinols cure at temperatures from 21°C to 66°C (70°F to 150°F)	High dry and wet strength; very resistant to moisture and damp atmospheres; more resistant than wood to high temperature and chemical aging.	Primary adhesives for laminated timbers and assembly joints that must withstand severe service conditions.
Urea	Powder and liquid forms; may be blended with melamine or other more durable resins; white to tan resin with colorless bondline	Powder mixed with water, hardener, filler, and extender by user; some formulations cure at room temperatures, others require hot pressing at 120°C (250°F); curable with high-frequency heating	High dry and wet strength; moderately durable under damp atmospheres; moderate to low resistance to temperatures in excess of 50°C (122°F)	Hardwood plywood; furniture; fiberboard; particleboard; underlayment; flush doors; furniture cores

must be cured at high temperatures and require expensive, heated presses. Some of these can be cured within minutes in expensive high frequency heated presses. Cold presses or clamps are satisfactory for room-temperature-curing adhesives, although the long curing time in production can be a constraint. Even after hot or cold pressing, adhesive bonds must remain undisturbed until most of the curing has occurred.

There are other important considerations, particularly in furniture and interior millwork, where appearance is all-important. Adhesive color, ability to absorb stains and finishes, and freedom from bleeding and staining are critical factors. The urea-formaldehyde and polyvinyl acetate adhesives used in the furniture industry are formulated to give a tan or colorless joint with good acceptance of stain.

Ease and simplicity of use can also be important factors. One-part adhesives, like liquid animal, polyvinyl acetate, hot-melt, and phenol-formaldehyde film, are the simplest to use because there is no chance for error in weighing and mixing components. Water-dispersed and film adhesives are easy to clean up, whereas films are the least messy. Two-, three-, or multiple-part adhesives require careful measuring and mixing of components. They often require special solvents for cleanup after bonding. Frequently, adhesives are toxic to the skin or give off toxic fumes. Formaldehyde hardener for resorcinol, phenol, melamine, and urea adhesives is an irritant to many people. Amine hardeners in some epoxy adhesives are strong skin sensitizers.

The cost of an adhesive and related application equipment must be balanced against comparable cost factors for substituted adhesives. In recent years, the cost of organic solvents and the cost of recovering volatiles to prevent air pollution have increased. Substituted water-based systems can be cheaper due to low cost of the solvent; however, grain raising of the wood and slower drying must be considered because of their effects on performance and overall cost.

Bonding Process

The bonding process involves a great number of factors that determine how successfully an adhesive bond will ultimately perform in service. The better these factors are understood and controlled, the fewer bonding problems will be encountered, along with their attendant expense. It is not necessary to be an adhesive expert to manufacture acceptable bonds, although more knowledge is always helpful. However, it is essential that the user follow the instructions of the adhesive supplier during the entire bonding process. The supplier has extensive technical knowledge of an adhesive's composition, working characteristics, and performance properties, which is reinforced by the experience of customers.

Moisture Content Control

After adhesive selection, the next most important factor contributing to trouble-free service of adhesive bonds is control of wood moisture content before and during the bonding process. Moisture content strongly affects the final strength and durability of joints, development of surface checks in the wood, and dimensional stability of the bonded assembly. Large changes in the moisture content compared with that at the time of bonding will cause shrinking or swelling stresses that can seriously weaken both wood and joints and can cause warping, twisting, and surface irregularities. Wood should not be bonded at high moisture content, particularly high density hardwoods that have large coefficients of shrinkage, unless the in-service moisture content is also expected to be high. The wood should be dry enough so that even if moisture is added during bonding, the moisture content is at about the level expected for the assembly in service.

Determining the proper moisture content for bonding depends primarily on the amount of moisture that is contained in the wood and adhesive and whether or not the adhesive curing process involves heating. For example, if boards are bonded at room temperature, the final moisture content is controlled mainly by the moisture content of the wood. In a lumber laminate, the number of bondlines are so few that a waterborne adhesive adds only 1% to 2% to the total moisture content of the laminate. However, if several pieces of veneer are bonded at room temperature, the moisture added by the adhesive in many bondlines can significantly increase the moisture content of the wood well above the target in-service level. Thus, thickness of the laminates, number of laminates, density of the wood, water content of the adhesive, quantity of adhesive spread, and hot or cold pressing all have a cumulative effect on the moisture content of the wood. During hot pressing, a moderate amount of water evaporates from the laminate as it is removed from the press. However, to minimize plastic flow of the hot and moist wood and prevent steam blisters or blows, the total moisture content of the assembly should not exceed 10% during hot pressing. A lumber moisture content of 6% to 7%, assuming 1% to 2% will be added by aqueous adhesives, is satisfactory for cold pressing of furniture and interior millwork. Lumber being laminated for exterior use should contain 10% to 12% moisture before bonding. A moisture content of 3% to 5% in veneer at the time of hot pressing is satisfactory for hardwood plywood intended for furniture and interior millwork and for softwood plywood intended for construction and industrial uses.

Lumber that has been kiln dried to the approximate average moisture content intended for bonding may still be at different moisture content levels between boards and within individual boards. Large differences in the moisture content between adjacent boards bonded together result in considerable stress on the common joint as the boards equalize toward a common moisture content. Best results are achieved when differences are not greater than about 5% for lower density species and 2% for high density species.

The moisture content of wood in bonded products should be targeted to the EMC that the product will experience in

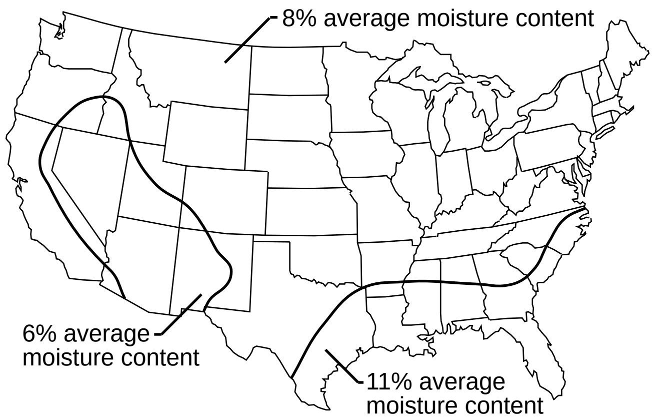

Figure 9–4. Average equilibrium moisture content for wood in building interiors in regions of the United States.

service. These conditions vary in the United States; regional average EMC values of wood in building interiors are shown in Figure 9–4. The average moisture content for most of the United States is 8%. The average increases to 11% along the Atlantic and Gulf coastal regions; in the arid southwest, the EMC is relatively low at 6%. The moisture content of wood exposed to outdoor air generally is higher and more variable and averages near 12% and ranges from 7% to 14% in most of the United States. During winter in the northern states, heating of indoor air that is normally dry lowers wood EMC to 4% to 5%. Furniture manufactured in the southeast at 11% EMC, then sold or moved to northern states where EMC drops to 4%, usually will experience some splitting, delamination of joints, or other noticeable appearance defects. Manufacturers of bonded wood products must be aware of these regional and seasonal variations, then condition the wood and bond it at moisture content levels that are consistent with regional service conditions.

Surface Preparation

The physical and chemical condition of wood and nonwood surfaces was described in a previous section where emphasis was placed on understanding the relationships between surface condition and adhesive bond performance. Wood surfaces are best prepared for maximum adhesive wetting, flow, and penetration by removing all materials that might interfere with bond formation to sound wood. Ideally, wood should be knife-planed within 24 h of adhesive spreading. However, other surfacing methods have been used successfully for certain types of bonded joints, including sawing for furniture and millwork, knife-cutting for veneer, and abrasive-planing for panels. All must produce smooth, flat, parallel surfaces, free from machining irregularities, such as burnishes, skips, and crushed, torn, and chipped grain. Properly planed flat surfaces help ensure that a layer of adhesive of uniform thickness can be uniformly spread over the adherend.

Adhesive Spreading

Regardless of method, the purpose in spreading adhesive is to distribute an adequate amount of adhesive of uniform thickness over the bonding area, so that under pressure, the adhesive will flow into a uniformly thin layer. Assuming that the spreader is capable of applying adhesive uniformly and that surfaces are smooth, flat, and parallel, then adhesive will flow ideally if uniform pressure is applied. The amount of adhesive needed will depend on the wood species, moisture content, type of adhesive, temperature and humidity of the air, assembly time, and whether adhesive will be applied to one or both surfaces. Adhesives can be spread by hand with brush, roller, or bead-extruder, but in manufacturing, adhesives are applied by machines, such as roll-spreader, extruder, curtain-coater, or spray. Instead of applying a uniform film, extruders apply continuous, uniformly spaced beads of discreet diameter and flow rate. When pressure is applied to both adherends, the adhesive is squeezed into a uniformly thin layer. An extruder of this type is used to apply adhesive to veneer in the manufacture of laminated veneer lumber (LVL) (Fig. 9–5). A pressurized extruder is used in the field to apply a single bead of elastomeric construction adhesive to joists for a plywood floor system (Fig. 9–6).

Assembly and Pressing

Control of consistency after the adhesive has been spread and until pressure is applied is a balancing act of a variety of factors. The relationships between adhesive consistency and bonding pressure as they affect formation of strong bonds are illustrated in Figure 9–7. Adhesive consistency strongly affects adhesive wetting, flow, and penetration, particularly the transfer of adhesive to an unspread wood surface, when pressure is applied to the assembly. Adhesive consistency depends upon type of adhesive, type of solvent, and proportion of solvent in the mixture, age of adhesive mixture, amount of adhesive spread, species of wood, moisture content of wood, temperature of wood, temperature and humidity of surrounding air, and the critically important evaporation and absorption of solvent during the assembly time. Assembly time is the time between spreading adhesive on wood surfaces and applying pressure to the assembly. When the adhesive-spread surfaces remain open before assembly (open assembly), then consistency is most affected by evaporative capacity of the surrounding air and absorbency of the wood's surface. When the assembly is closed and before applying pressure (closed assembly), consistency is most influenced by absorbency factors and least affected by evaporation. Cold-setting waterborne wood adhesives lose water by absorption and evaporation so that consistency steadily increases until they eventually set. Thermosetting waterborne adhesives also dry out, but despite water loss, they flow to some extent in the presence of heat, then harden with additional heating.

Pressure serves several useful purposes: it forces entrapped air from the joint; it brings adhesive into molecular contact with the wood surfaces; it forces adhesive to penetrate into the wood structure for more effective mechanical interlocking;

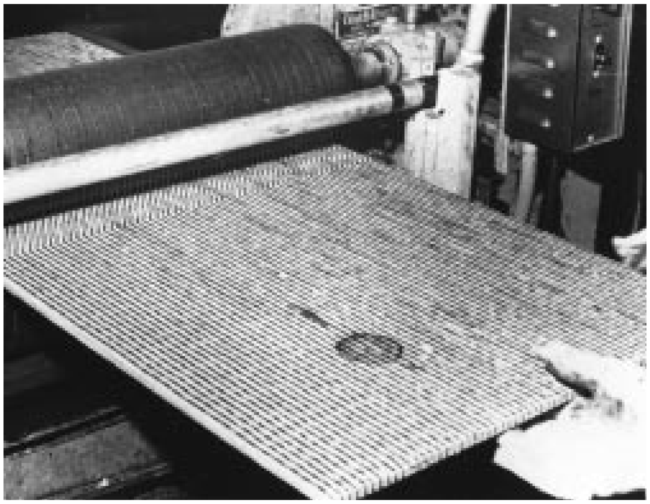

Figure 9–5. An extruder applies continuous and uniformly sized and spaced beads of adhesive to veneer for laminating into LVL.

Figure 9–6. A pressurized extruder applies a single bead of elastomeric construction adhesive to floor joists for assembly of a plywood floor system.

it squeezes the adhesive into a thin continuous film; and it holds the assembly in position while the adhesive cures. But if pressure is too high, the adhesive can overpenetrate porous woods and cause starved joints that are inferior in bond strength (Fig. 9–7). The strongest joints result when the consistency of the adhesive permits the use of moderately high pressures that are consistent with the recommended pressures for the density of the wood.

Low pressures near 700 kPa (100 lb/in^2) are suitable for low density wood because the surfaces easily conform to each other, thus ensuring intimate contact between adhesive and

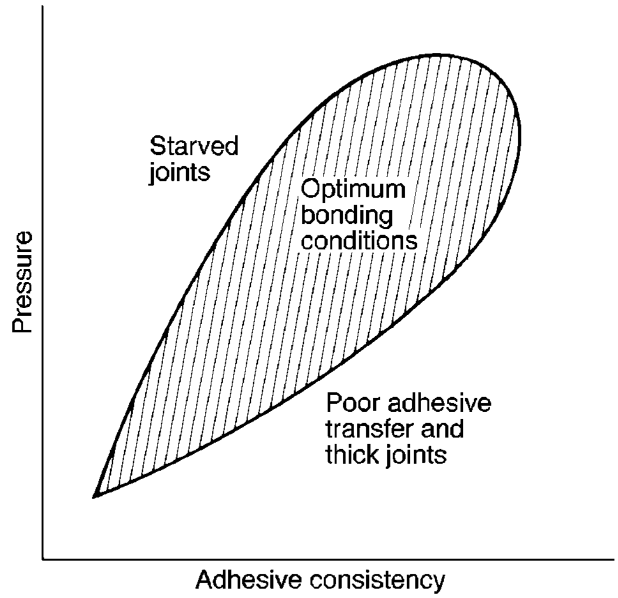

Figure 9–7. An illustration of the relationships between adhesive consistency and bonding pressure as they affect bond formation by a thermosetting adhesive.

wood. High pressures up to 1,700 kPa (247 lb/in^2) are required for the highest density woods that are difficult to compress. Flat, well-planed surfaces of small area can be bonded satisfactorily at lower pressures; however, because high pressure tends to squeeze adhesive into the wood or out of the joint, adhesives of greater consistency are required for denser woods (Fig. 9–7). Greater consistency can be achieved with longer assembly time, which allows increased absorption of liquid solvent by the wood and evaporation into the air. Care is required, regardless of wood density, to ensure that the assembly time is not excessive, lest the adhesive dry out or even precure before pressure is applied. Predried or precured adhesive will result in inadequate transfer of adhesive to an opposite unspread surface, and the bondline will be thick and weak (Fig. 9–7).

Lumber joints should be kept under pressure until they have enough strength to withstand handling stresses that tend to separate the pieces of wood. When cold-pressing lumber under normal bonding conditions, this stage can be reached in as little as 15 min or as long as 24 h, depending on the temperature of the room and the wood, the curing characteristics of the adhesive, and the thickness, density, and absorptive characteristics of the wood. When hot pressing, the time under pressure varies with temperature of platens, thickness and species of wood, and adhesive formulation. In actual practice, hot-pressing times vary from 2 to 15 min and up to 30 min for very thick laminates. The time under pressure can be reduced to less than 3 min with high frequency heating. High frequency concentrates energy in the conductive bondline to rapidly cure the adhesive. It is commonly used for bonding lumber, forming end- and edge-grain joints, patching, scarfing, fingerjointing plywood, and in manufacturing various panel products. Careful control of power and press time is essential to prevent formation of steam that could lead to steam blows and even arcing.

It has been observed that bondlines of structural adhesives that withstand the highest of stresses from mechanical loading and dimensional changes generally have bondline thicknesses within the range of 0.076 to 0.152 mm (0.003 to 0.006 in.). Below this range, the bondlines are too thin to effectively transfer stresses from one adherend to the other, particularly stresses from moisture-induced dimensional changes. Above this range, bond strength becomes progressively weaker as bondline thickness increases. Structural wood adhesives are brittle, so they fracture more in thicker bondlines than in thinner ones. These adhesives also contain solvents, and because solvent is lost while curing, the thicker adhesive film shrinks and fractures more than the thinner and may contain more voids from entrapped solvent gases. Thick bondlines result from inadequate pressure, either from low applied pressure or from rough, uneven, poorly mated surfaces. When uneven surfaces are joined, pressure will not be uniform along the bondline. As a result, the adhesive will be squeezed out from the areas of very high pressure, and the areas of little to no pressure will have very thick bondlines. Both starved and thick bondlines produce weak joints.

Post-Cure Conditioning

In the process of bonding edge-grain joints, the wood in the joint absorbs moisture from the adhesive, then swells. If the bonded assembly is surfaced before this excess moisture is evaporated or absorbed uniformly, more wood is removed along the swollen joint than elsewhere. Later, when the added moisture evaporates, the wood in the joint shrinks beneath the surface. These sunken bondlines become very conspicuous under a high-gloss finish. This is a particularly important consideration when using adhesives that contain relatively large amounts of water. Redistribution of moisture added by the adhesive can be accomplished by conditioning the bonded assembly for 24 h at 70°C (158°F), 4 days at 50°C (122°F), or at least 7 days at room temperature before surfacing. In each case, the relative humidity must be adjusted to prevent drying the wood below the target moisture content.

After bonding, plywood-type constructions should be conditioned to the average moisture content expected in service. The best conditioning is accomplished by controlling humidity on time schedules. If bonded products cured at room temperature are exposed to excessively low moisture content, warping, checking, and opening of joints will increase significantly. Softwood plywood is very dry after hot pressing, so panels may be sprayed with water and tightly stacked to allow moisture to diffuse uniformly. This practice restores some of the panel thickness lost by compression during hot pressing and apparently minimizes warping in service.

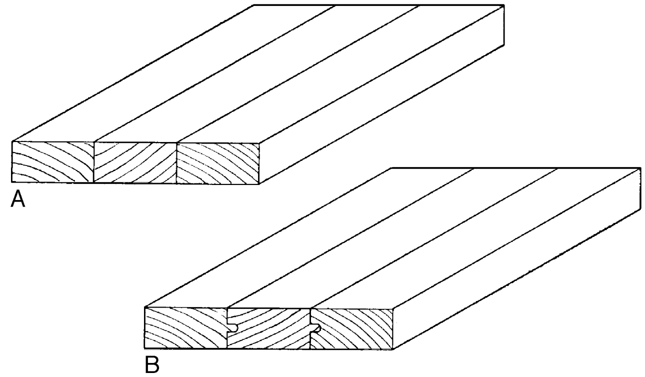

Figure 9–8. Edge-grain joints: A, plain; B, tongue-and-groove.

Bonded Joints

Edge-Grain Joints

Face-grain joints (wide surface of a board) are commonly seen in structural laminated lumber products, where adhesive bonds are stronger than the wood. Edge-grain joints (narrow surface of a board) (Fig. 9–8) can be almost as strong as the wood in shear parallel to the grain, tension across the grain, and cleavage. The tongue-and-groove joint (Fig. 9–8) and other shaped edge-grain joints have a theoretical strength advantage because of greater surface area than the straight, edge-grain joints, but they do not produce higher strength. The theoretical advantage is lost, wholly or partly, because the shaped sides of the two mating surfaces cannot be machined precisely enough to produce the perfect fit that will distribute pressure uniformly over the entire joint area. Because of poor contact, the effective bonding area and strength can actually be less in a shaped joint than on a flat surface. The advantage of the tongue-and-groove and other shaped joints is that the parts can be more quickly aligned in clamps or presses. A shallow-cut tongue-and-groove is just as useful in this respect as a deeper cut, and less wood is wasted.

End-Grain Joints

It is practically impossible to make end-grain butt joints (Fig. 9–9) sufficiently strong to meet the requirements of ordinary service with conventional bonding techniques. Even with special techniques, not more than about 25% of the tensile strength of the wood parallel-to-grain can be obtained in a butt joint. To approximate the tensile strength of clear solid wood, a scarf joint or fingerjoint must closely approach the parallel-to-grain direction of the edge-grain joint (Fig. 9–8). The surface area of this edge-grain joint should be at least 10 times greater than the cross-sectional area of the piece, because wood is approximately 10 times stronger in tension than in shear. In plywood scarfs and fingerjoints, a slope of 1 in 8 is typical for structural products. For non-structural, low-strength joints, these requirements need not be met.

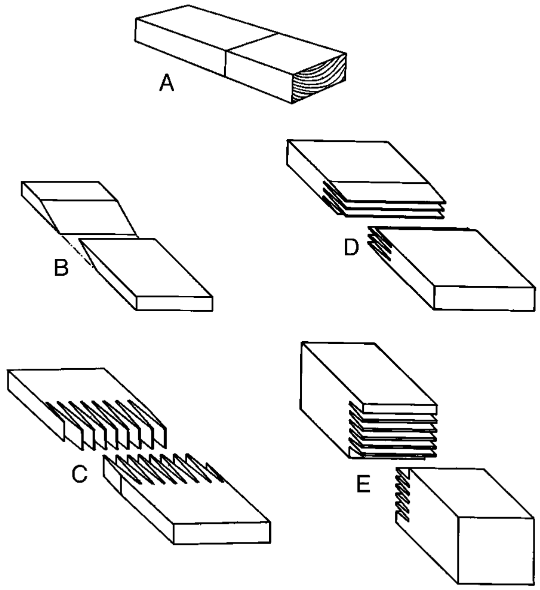

Figure 9–9. End-grain joints: A, butt; B, plain scarf; C, vertical structural fingerjoint; D, horizontal structural fingerjoint; E, nonstructural fingerjoint.

Fingerjoints can be cut with the profile showing either on the wide face (vertical joint) or on the edge (horizontal joint) (Fig. 9–9). There is greater area for designing shapes of fingers in the vertical joint, but a longer cutting head with more knives is needed. When the adhesive is cured by high frequency heating, the cure is more rapid with the vertical than with the horizontal joint. A nonstructural fingerjoint, with fingers much shorter than in the two structural fingerjoints, is shown in Figure 9–9.

A scarf joint is shown in Figure 9–9. Slopes of 1 in 12 or flatter produce the highest strength. This is also true in fingerjoints, but the tip thickness must be small and no greater than 0.8 mm (0.031 in.). A thickness of 0.4 to 0.8 mm (0.016 to 0.031 in.) is about the practical minimum for machined tips. Sharper tips can be created with dies, which are forced into the end grain of the board.

A well-manufactured end joint of either scarf, finger, or lap type can have up to 90% of the tensile strength of clear wood and exhibit behavior much like that of clear wood. However, test results indicate that the cycles-to-failure for a well-manufactured end joint are somewhat lower compared with the results of similar tests for clear wood.

End-to-Edge-Grain Joints

Plain end-to-edge-grain joints (Fig. 9–10) are difficult to design to carry appreciable loading. Furthermore, internal stresses develop in the members in service from unequal dimensional changes with moisture content changes. Such stresses can be great enough to cause failure. As a result, it is necessary to design these joints with interlocking

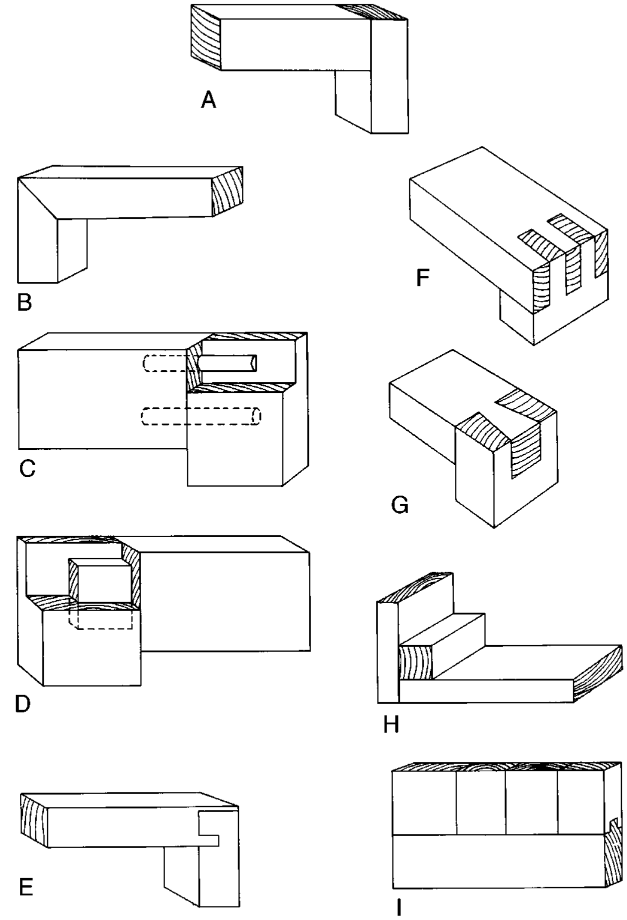

Figure 9–10. End-to-edge-grain joints: A, plain; B, miter; C, dowel; D, mortise and tenon; E, dado tongue and rabbet; F, slip or lock corner; G, dovetail; H, blocked; I, tongue-and-groove.

surfaces, for example, dowels, mortise and tenons, rabbets (Figs. 9–10), so that edge grain of the interlocking piece is bonded to the edge grain of the adjoining piece. The joint area is enlarged as well. All end-to-edge-grain joints should be protected from appreciable changes in moisture content in service.

Construction Joints

Elastomeric construction adhesives are commonly used in the light-frame construction industry for field assembly of panelized floor and wall systems. Structural panels are bonded to floor joists and wall studs with mastic adhesives that have the unique capability of bridging gaps up to 6.5 mm (0.25 in.) between rough and poorly fitting surfaces (Fig. 9 –11). Without any premixing, the adhesive is extruded in a bead along framing members with a hand-held caulking gun or a pressurized dispenser similar to that shown in Figure 9–6. Nails or screws provide the only pressure for bonding, and they hold materials in position while the adhesive sets. Elastomerics are also uniquely tolerant of the

Figure 9–11. Gap-filling construction adhesive in field-assembled plywood floor system.

temperature and moisture content variations at field construction sites. Although they do not deliver the strength and durability of conventional structural adhesives, elastomerics are strong and flexible enough to give long-term performance under most conditions of installation and service.

Construction adhesives enable a nailed floor system to act to some degree as a composite assembly with increased stiffness. Greater stiffness permits joists to be longer and spaced more widely, with one layer of plywood subflooring replacing two. Floors are less bouncy with fewer squeaks and nail pops. However, structural design of the composite assembly is based only on the increased stiffness of nailed panel and framing materials. Structural credit for strength is not allowed for the adhesive in the engineering design.

Testing and Performance

An adhesive is expected to hold materials together and transfer design loads from one adherend to the other within a given service environment for the life of the structure. The purpose of testing performance is to ensure that adhesive bonds will not deteriorate before they can meet these expectations. A variety of methods are available to test bonding performance, particularly for bonded assemblies. Generally, these testing methods attempt to predict how bonded joints are likely to perform in a specific loading mode (shear, tensile, creep) in an assembly at specific temperature and moisture conditions for a specific time.

Most performance tests are short term. They are based on chemical, mechanical, and rheological laboratory tests of adhesive polymers and their adhesives and bonds. Intermediate-term tests of products that are conducted in pilot operations and field experiments are integrated with short-term laboratory tests in an effort to extrapolate these data into long-term performance. Long-term tests of bonded assemblies under actual environmental exposures are conducted, but these supporting data may not be available for 10 to 30 years. Therefore, heavy reliance must be placed on short-term tests to predict long-term performance. As the relationships between chemical structure and mechanical performance, particularly long-term performance, are better understood, the greater the reliance will be on short-term testing.

Analytic Chemical and Mechanical Testing of Polymers

The molecular structures of adhesive polymers are chemically characterized spectroscopically by nuclear magnetic resonance, either in the liquid or solid state. Molecular-size distributions of polymers are determined by gel permeation chromatography. Rates of chemical reaction are studied by differential scanning calorimetry. The rheological properties of curing and cured adhesives are characterized by dynamic mechanical analysis and torsional-braid analysis. Sophisticated fracture mechanics techniques are used to measure toughness of adhesive bonds as they fail in a cleavage mode. High magnification microscopes, including scanning electron microscope, transmission electron microscope, and atomic force microscope, enable scientists to visually analyze surfaces of adhesives and adherends before, during, and after fracture. Much can be learned from measurements of chemical, mechanical, and rheological properties of polymers and adhesives as they exist apart from adherends. Until such data can be correlated with performance, there is no substitute for testing performance in bonded assemblies prepared with specific adhesives and materials, and tested under specific loading modes, environmental conditions, and duration of loading. When adhesives are formulated through a blend of scientific analysis and art of formulation, they are tested for strength and durability in the laboratory and field, usually by industry- and government-accepted standard methods of test and product specifications.

Mechanical Testing of Bonded Assemblies

Responses of adhesive-bonded assemblies to mechanical loading are defined in terms of several commonly used modes of applying stress to joints. In all test modes, specific materials, conditions of materials and test, and testing procedures are completely specified to ensure repeatability to enable valid comparisons of data. Most test methods, specifications, and practices for adhesives and bonded assemblies are consensus standards published each year in the *Annual Book of ASTM Standards* by the American Society for Testing and Materials (ASTM). Several trade associations have their own specifications and performance standards that apply to their specific wood products. The Federal government also has specifications that are used by the General Services Administration to purchase products.

Four basic stressing modes—shear, tensile, cleavage, and peel—are commonly used to test bonded wood assemblies with variations of these to determine strength levels during impact, flexure, fatigue, and creep under long-term stress.

The following describes the basic stress modes in adhesive-bonded joints:

- Shear, resulting from forces applied parallel to the bondline
- Tensile, resulting from forces applied perpendicular to the bondline
- Cleavage, resulting from separation along a bondline by a wedge or other crack-opening type of force
- Peel, resulting from forces applied to a bondline that tend to progressively separate a flexible member from a rigid member or another flexible member

As the names imply, impact, fatigue, and creep are tests that have more to do with the rate at which basic modes and variations are applied. Impact loads are sudden, much faster relative to the controlled slow rates of shear or tensile stressing. Fatigue is the loss in strength from repeated loading and reflects deterioration of bonds from mechanical rather than environmental stresses, although the latter may be imposed during fatigue testing. Creep loads are statically applied but are of prolonged duration that can last from a few days to years, usually at extreme conditions of environmental exposure. The flexure test applies a bending force to a simple beam at midspan, perpendicular to the bondline. In a laminated beam, the test directs a large proportion of the shear forces to bondlines between the laminates.

The common measures used to estimate potential performance of bonded wood joints are strength, wood failure, and delamination. Best performance produces a bond strength that is greater than that of the wood, wood failure that is more than 75% over the bonded area, and delamination of the joint that is less than 5% for softwoods and 8% for hardwoods, under severe service conditions. These performance values reflect how wood adherend, adhesive bond, and environmental exposure have interacted in response to loading.

Bond strength is tested most commonly in shear parallel to the grain. Because most wood adhesives exceed the shear strength of wood in this direction, the maximum potential strength of the adhesive may not be realized, particularly for moderate to lower density species. Bonds in structural assemblies are expected to exceed the strength of the wood, so in traditional design of joints, adhesive strength has been ignored. Adhesives not as strong as wood simply have not been used in design because methods for determining allowable mechanical properties of adhesives for engineering design had not been developed. One such method now exists as a consensus standard—ASTM D5574–97 (ASTM 1997). Exceeding the strength of wood is an essential performance criterion; therefore, the amount of wood that fails in a joint is estimated as a percentage of the area of the bonded joint. This is an important indicator of bond strength, often more important than the measured shear strength of the bond. The higher the wood failure and the deeper the fracture into the grain of the wood, the stronger and more durable the bond, particularly with durable types of adhesives. If wood failure is shallow with only wood fibers remaining attached to the adhesive film, then bond strength and probably durability is lacking in the bond. Thus, a consistently high level of wood failure, above 75% in some standards and above 85% in others, means that shear strength associated with these average wood failures are good estimates of the load-carrying capability of the joint. High levels of wood failure and shear strength in a wet and hot environment might indicate that the adhesive bond is as strong as the wood. If cycles of alternate drying were included with cycles of wet and hot conditions, then high wood failure would indicate even more durable bonds. High wood failure in shear tests of water-saturated bonds is also a strong indicator of bond durability, particularly with durable types of adhesives. Wood failure is considered a valid measure of bond strength only to solid wood, not to reconstituted products made of bonded wood particles.

High shear strength and wood failure in themselves are not sufficient indicators of the durability of a structural bond. Delamination is an indicator of how well the bonded joint withstands severe swelling and shrinking stresses in the presence of high moisture and heat. Delamination is the separation between laminates because of adhesive failure, either in the adhesive or at the interface between adhesive and adherend. If strength of adhesion is not as strong as the wood in resisting forces tending to separate laminates, then delamination occurs. If adhesion does resist delaminating forces, then the wood will fail adjacent to the bondline, but not within the adhesive. The stressing modes induced by stresses from moisture-related dimensional changes are combinations of tensile and shear forces with cleavage acting at joint edges. Delamination of adhesives in structural laminated wood products exposed to the cyclic delamination test in ASTM D2559–97 (ASTM 1997) cannot exceed 5% in softwoods and 8% in hardwoods.

Short- and Long-Term Performance

In the short term, the mechanical properties of wood, adhesives, and bonded products vary with the specific environmental exposure. In most cases, all properties decrease as the temperature and moisture levels increase. Strength and stiffness may return to their original levels if the yield points of the materials have not been exceeded while under load. The properties of rigid thermosetting adhesives like resorcinol-formaldehyde, phenol-formaldehyde, melamine-formaldehyde, and urea-formaldehyde change less than do wood properties under equivalent temperature and moisture changes. Therefore, evaluating short-term performance of products made with these adhesives is simply a matter of testing bonds at room temperature in dry and wet conditions. Thermoplastic adhesives like casein, polyvinyl acetate, and elastomerics, whose properties change more rapidly than those of wood with changes in moisture and heat, are tested dry, dry after water soaking, and after prolonged exposure to high humidity environments. In addition, some specifications require testing of bonded structural and nonstructural products at elevated temperatures such as occur in roofs or enclosed shipping containers. A short-term dead-load test at elevated temperatures may also be required. Specifications for adhesives for structural products like laminated beams and plywood require conformance to high minimum strength and wood failure values after several different water exposure tests. Adhesive bonds in laminated beams must also withstand severe cyclic moisture content and temperature changes, with only low levels of delamination allowable.

In the long term, wood, adhesives, and bonded products deteriorate at a rate determined by the levels of temperature, moisture, stress and, in some instances, by concentrations of certain chemicals and presence of microorganisms. Long-term performance is equated with the ability of a product to resist loss of a measured mechanical property over the time of exposure. A durable product is one that shows no greater loss of properties during its life in service than wood of the same species and quality.

Many adhesives in bonded products have decades of documented performance in many environments. Thus, it is possible to predict with a high degree of certainty the long-term performance of similar products. Well-designed and well-made joints with any of the commonly used woodworking adhesives will retain their strength indefinitely if the moisture content of the wood does not exceed approximately 15% and if the temperature remains within the range of human comfort. However, some adhesives deteriorate when exposed either intermittently or continuously to temperatures greater than 38°C (100°F) for long periods. Low temperatures seem to have no significant effect on strength of bonded joints.

Products made with phenol-formaldehyde, resorcinol-formaldehyde, and phenol-resorcinol-formaldehyde adhesives have proven to be more durable than wood when exposed to warm and humid environments, water, alternate wetting and drying, and even temperatures sufficiently high to char wood. These adhesives are entirely adequate for use in products that are exposed indefinitely to the weather (Fig. 9–12).

Products well-made with melamine-formaldehyde, melamine-urea-formaldehyde, and urea-formaldehyde resin adhesives have proven to be less durable than wood. Melamine-formaldehyde is only slightly less durable than phenol-formaldehyde or resorcinol-formaldehyde, but it is still considered acceptable for structural products. Although considered less durable, melamine-urea-formaldehyde is also accepted in structural products at a melamine-to-urea ratio of 60:40. Urea-formaldehyde resin is susceptible to deterioration by heat and moisture (Fig. 9–12).

Products bonded with polyvinyl acetate and protein-based adhesives will not withstand prolonged exposure to water or repeated high–low moisture content cycling in bonds of high density woods. However, if they are properly formulated, these adhesives are durable in a normal interior environment.

Some isocyanate, epoxy, polyurethane, and cross-linked polyvinyl acetate adhesives are durable enough to use on lower density species even under exterior conditions, but exterior exposure must be limited for most of these. Some elastomer-based adhesives may be durable enough for limited exposure to moisture with lower density species in

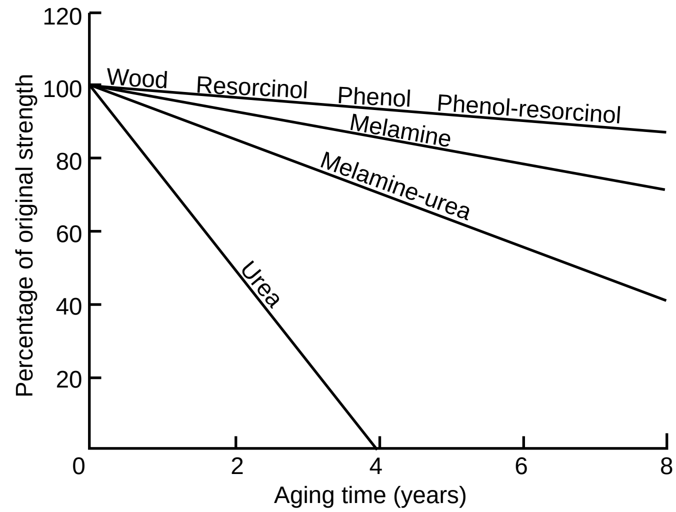

Figure 9–12. Relative rates of deterioration of bond strength of small specimens exposed directly to weather.

nonstructural applications or in structural applications when used in conjunction with approved nailing schedules. Polyurethane adhesives that chemically cure and still remain flexible are among the most durable construction adhesives.

New adhesives do not have a history of long-term performance in service environments, so accelerated laboratory exposures that include cycles of heat, moisture, and stress are used to estimate long-term performance. However, laboratory exposures cannot duplicate the actual conditions of a service environment. Estimates of long-term performance can be obtained by exposing specimens outdoors for up to 30 years. Outdoor exposures may be intensified by facing specimens south at an angle perpendicular to the noonday sun and by establishing exposure sites in regions with the most extreme of service environments, for example, southern coastal and arid southwestern regions. Only four long-term laboratory aging methods have been standardized, and none specifies minimum performance levels. Therefore, performance of any new adhesive or bonded product must be compared with the performance of established adhesives or products tested in the same laboratory exposure.

Product Quality Assurance

After the short- and long-term performance of a product has been established, then maintenance of the manufacturing process to ensure that the product will be made and perform at that level is the major concern of a quality-assurance program that consists of three parts:

1. Establishing limits on bonding process factors that will ensure acceptable joints and product.
2. Monitoring the production processes and quality of bond in joints and product.
3. Detecting unacceptable joints and product, determining the cause, and correcting the problem.

The structural panel, laminated-beam, particleboard, millwork, and other industrial trade associations have established quality-assurance programs that effectively monitor the joint and product performance at the time of manufacture for compliance with voluntary product standards. Usually, product performance is evaluated immediately after manufacture by subjecting specimens from the product to a series of swell–shrink cycles. The treatments are more rigorous for products intended for exterior exposure. For example, exterior softwood plywood is subjected to two boil–dry cycles, while interior plywood is subjected to a single soak–dry cycle at room temperature. After exposure, the specimens are examined for delamination or evaluated for percentage wood failure. The test results are compared with the minimum requirement in the trade association's standards. Lengthy experience and correlations between exterior performance and accelerated laboratory tests have shown that products with at least the minimum values will probably perform satisfactorily in service. If the product meets the requirement, it is certified by the association as meeting the standard for satisfactory performance.

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Chapter 10

Wood-based Composites and Panel Products

John A. Youngquist

Contents

Because wood properties vary among species, between trees of the same species, and between pieces from the same tree, solid wood cannot match reconstituted wood in the range of properties that can be controlled in processing. When processing variables are properly selected, the end result can sometimes surpass nature's best effort. With solid wood, changes in properties are studied at the cellular level. With reconstituted wood materials, changes in properties are studied at the fiber, particle, flake, or veneer level. Properties of such materials can be changed by combining, reorganizing, or stratifying these elements.

The basic element for composite wood products may be the fiber, as it is in paper, but it can also be larger wood particles composed of many fibers and varying in size and geometry. These characteristics, along with control of their variations, provide the chief means by which materials can be fabricated with predetermined properties.

In any discussion of the strength properties of wood-based panels and other adhesive-bonded wood composites, the first consideration is the constituents from which these products are made (O'Halloran and Youngquist 1984; Youngquist 1987, 1988). The basic wood elements that can be used in the production of wood-based panels are shown in Figure 10–1. The elements can be made in a great variety of sizes and shapes and can be used alone or in combination. The choice is almost unlimited.

Currently, the term composite is being used to describe any wood material adhesive-bonded together. This product mix ranges from fiberboard to laminated beams and components. Table 10–1 shows a logical basis for classifying wood composites proposed by Maloney (1986). For the purposes of this chapter, these classifications were slightly modified from those in the original version to reflect the latest product developments. Composites are used for a number of structural and nonstructural applications in product lines ranging from panels for interior covering purposes to panels for exterior uses and in furniture and support structures in many different types of buildings.

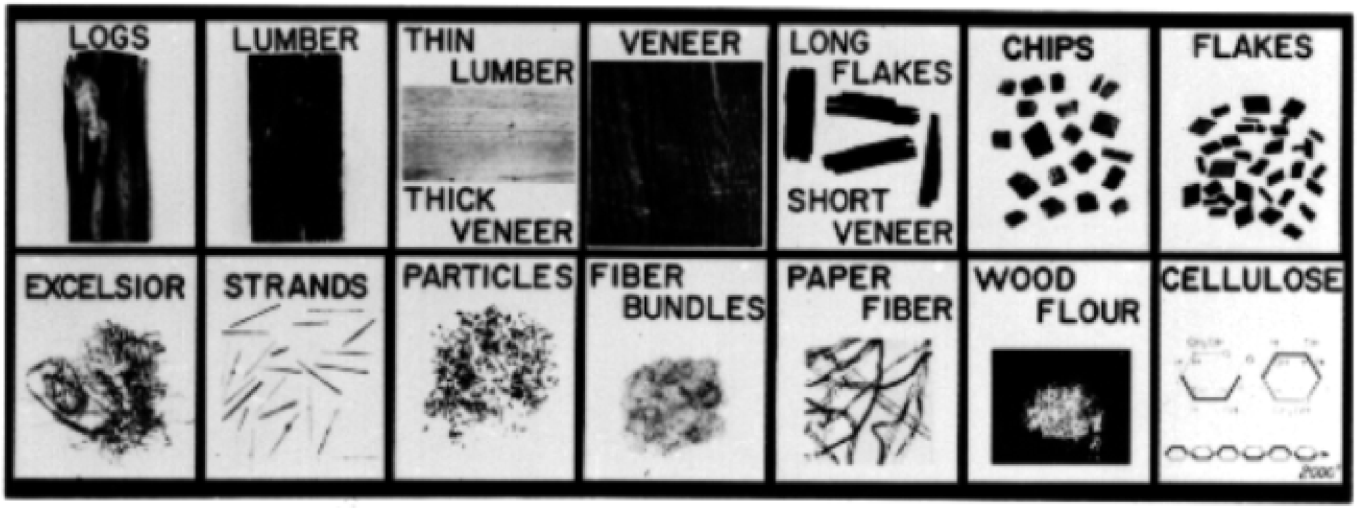


Figure 10–1. Basic wood elements, from largest to smallest (Marra 1979).

Table 10–1. Classification of wood-based composites[a]

Veneer-based material
Plywood
Laminated veneer lumber (LVL)
Parallel-laminated veneer (PLV)
Laminates
Laminated beams
Overlayed materials
Wood–nonwood composites[b]
Composite material
Cellulosic fiberboard
Hardboard
Particleboard
Waferboard
Flakeboard
Oriented strandboard (OSB)
COM-PLY[c]
Edge-adhesive-bonded material
Lumber panels
Components
I-beams
T-beam panels
Stress-skin panels
Wood–nonwood composites
Wood fiber–plastic composites
Inorganic-bonded composites
Wood fiber–agricultural fiber composites

[a]Maloney 1986.
[b]Panels or shaped materials combined with nonwood materials such as metal, plastic, and fiberglass.
[c]Registered trademark of APA–The Engineered Wood Association.

Figure 10–2 provides a useful way to further classify wood-based composite materials. This figure presents an overview of the most common types of products discussed in this chapter as well as a quick reference to how these composite materials compare to solid wood from the standpoint of density and general processing considerations. The raw material classifications of fibers, particles, and veneers are shown on the left *y* axis. Specific gravity and density are shown on the top and bottom horizontal axes (*x* axes). The right *y* axis, wet and dry processes, describes in general terms the processing method used to produce a particular product. Note that both roundwood and chips can serve as sources of fiber for wet-process hardboard. Roundwood or wood in the form of a waste product from a lumber or planing operation can be used for dry-processed products. For medium-density fiberboard (MDF), resin is usually applied to the fiber after the fiber is released from the pressurized refiner. The fiber is then dried, formed into a mat, and pressed into the final product. For other dry-processed products, the material is fiberized and dried and then adhesive is added in a separate operation prior to hot pressing into the final composite product. Figure 10–3 shows examples of some composite materials that are represented in schematic form in Figure 10–2.

Scope

Although there is a broad range of wood composites and many applications for such products, for the purposes of this chapter, wood composites are grouped into three general categories: plywood, particle and fiber composites, and wood–nonwood composites. Books have been written about each of these categories, and the constraints of this chapter necessitate that the discussion be general and brief. References are provided for more detailed information. Information on adhesive-bonded-laminated (glulam, timbers, and structural composite lumber, including laminated veneer lumber) and adhesive-bonded members for lumber and panel products

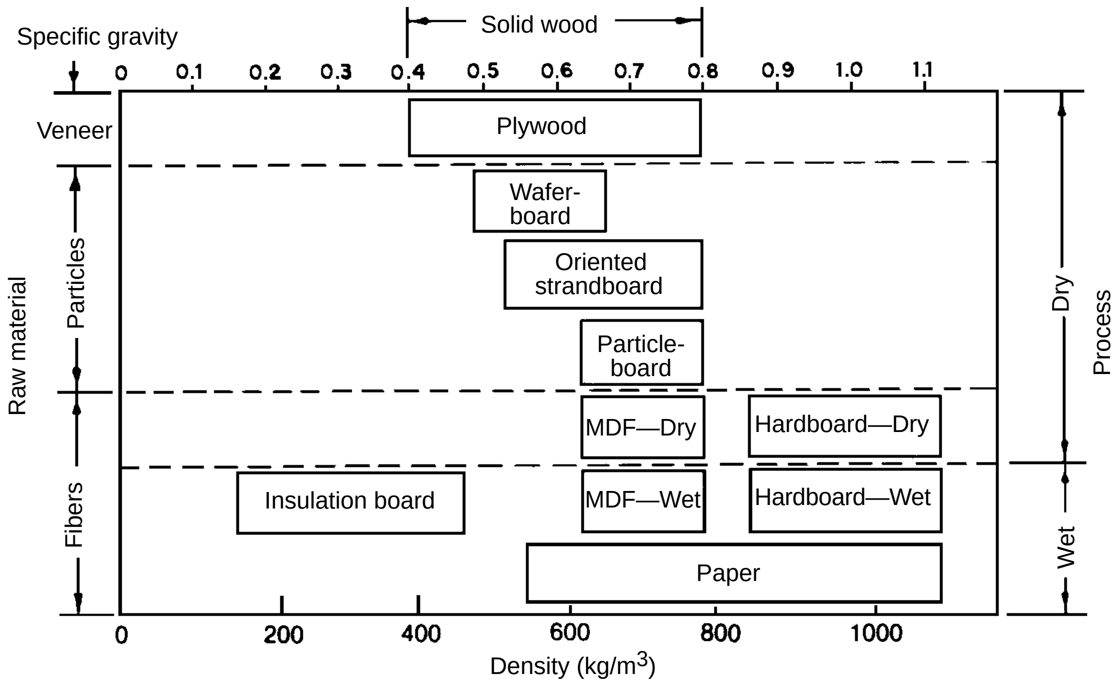


Figure 10–2. Classification of wood composite boards by particle size, density, and process type (Suchsland and Woodson 1986).

is presented in Chapter 11 of this handbook. Many composite materials, like fiberboard, MDF, and particleboard, can be made from wood alone or in combination with agricultural fibers (Youngquist and others 1993a, 1994; Rowell and others 1997).

The first category, plywood, is covered in some detail because the process for manufacturing this kind of material is quite different from that used for other composite materials and because there are many different classes and grades of plywood in the marketplace. The second category, composite materials, includes oriented strandboard (OSB), particleboard, and fiberboard. These types of composites undergo similar processing steps, which are discussed in general terms for all the products in the Particle and Fiber Composites section. The first and second categories of composite materials are further generally classified as conventional composite materials. The third category, wood–nonwood composites, includes products made from combining wood fibers with agricultural fibers, with thermoplastics, and with inorganic materials.

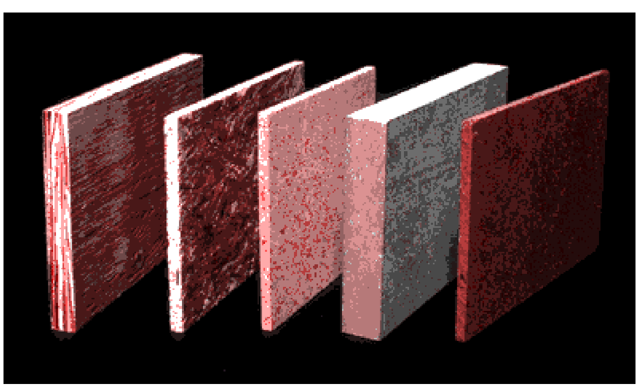


Figure 10–3. Examples of various composite products. From left to right: plywood, OSB, particleboard, MDF, and hardboard.

Types of Conventional Composite Materials

Conventional wood composite materials fall into five main categories based on the physical configuration of the wood used to make the products: plywood, oriented strandboard, particleboard, hardboard, and cellulosic fiberboard. Within limits, the performance of a conventional type of composite can be tailored to the end-use application of the product. Varying the physical configuration of the wood and adjusting the density of the composites are just two ways to accomplish this. Other ways include varying the resin type and amount and incorporating additives to increase water or fire resistance or to resist specific environmental conditions.

Adhesive Considerations

The conventional wood-based composite products discussed in this chapter are typically made with a thermosetting or heat-curing resin or adhesive that holds the lignocellulosic (wood) fiber together. The physical and mechanical properties of wood-based veneer, fiber, and particle panel materials are determined by standard American Society for Testing and Materials (ASTM) test methods. Commonly used resin–binder systems include phenol-formaldehyde, urea-formaldehyde, melamine-formaldehyde, and isocyanate.

Phenol-formaldehyde (PF) resins are typically used in the manufacture of products requiring some degree of exterior exposure durability, for example, OSB, softwood plywood, and siding. These resins require longer press times and higher press temperatures than do urea-formaldehyde resins, which results in higher energy consumption and lower line speeds (productivity). Products using PF resins (often referred to as phenolics) may have lowered dimensional stability because of lower moisture contents in the finished products. The inherently dark color of PF resins may render them unsuitable for decorative product applications such as paneling and furniture.

Urea-formaldehyde (UF) resins are typically used in the manufacture of products where dimensional uniformity and surface smoothness are of primary concern, for example, particleboard and MDF. Products manufactured with UF resins are designed for interior applications. They can be formulated to cure anywhere from room temperature to 150°C (300°F); press times and temperatures can be moderated accordingly. Urea-formaldehyde resins (often referred to as urea resins) are more economical than PF resins and are the most widely used adhesive for composite wood products. The inherently light color of UF resins make them quite suitable for the manufacture of decorative products.

Melamine-formaldehyde (MF) resins are used primarily for decorative laminates, paper treating, and paper coating. They are typically more expensive than PF resins. MF resins may be blended with UF resins for certain applications (melamine urea).

Isocyanate as diphenylmethane di-isocyanate (MDI) is commonly used in the manufacture of composite wood products; MDI is used primarily in the manufacture of OSB. Facilities that use MDI are required to take special precautionary protective measures.

These adhesives have been chosen based upon their suitability for the particular product under consideration. Factors taken into account include the materials to be bonded together, moisture content at time of bonding, mechanical property and durability requirements of the resultant composite products, and of course, resin system costs.

Some natural options may someday replace or supplement these synthetic resins. Tannins, which are natural phenols, can be modified and reacted with formaldehyde to produce a satisfactory resin. Resins have also been developed by acidifying spent sulfite liquor, which is generated when wood is pulped for paper. In the manufacture of wet-process fiberboard, lignin, which is inherent in lignocellulosic material, is frequently used as the resin (Suchsland and Woodson 1986).

Except for two major uncertainties, UF and PF systems are expected to continue to be the dominant wood adhesives for lignocellulosic composites. The two uncertainties are the possibility of much more stringent regulation of formaldehyde-containing products and the possibility of limitations to or interruptions in the supply of petrochemicals. One result of these uncertainties is that considerable research has been conducted in developing new adhesive systems from renewable resources.

Additives

A number of additives are used in the production of conventional composite products. One of the most notable additives is wax, which is used to provide finished products with resistance to aqueous penetration. In particle- and fiberboard products, wax emulsion provides excellent water resistance and dimensional stability when the board is wetted. Even small amounts (0.5% to 1%) act to retard the rate of liquid water pickup. These improved water penetration properties are important for ensuring the success of subsequent secondary gluing operations and for providing protection upon accidental wetting to the product during and after construction. The water repellency provided by the wax has practically no effect upon dimensional changes or water adsorption of composites exposed to equilibrium conditions. Other additives used for specialty products include preservatives, fire retardants, and impregnating resins.

General Manufacturing Issues

Successful manufacture of any composite wood product requires control over raw materials. Ideally, raw materials are uniform, consistent, and predictable. Wood does not offer these qualities but instead varies widely between species. For the purpose of producing a composite product, uniformity, consistency, and predictability are accomplished by reducing separated portions of the wood into small, relatively uniform and consistent particles, flakes, or fibers where effects of differences will average out. Size reduction is sometimes augmented by chemical treatments designed to weaken the bonds between the components. The degree of size reduction and the shape of individual lignocellulosic components will depend on the application. Different composites tolerate or demand different sizes and shapes. Generally speaking, all the conventional composite products discussed in this chapter are made to conform to product or performance standards (English and others 1997).

Standards for Wood–Based Panels

The general types of standards for panel products are product standards and performance standards. Table 10–2 lists standards for common conventional composite products. The term adhesive, as used in the following descriptions of product and performance standards, is synonymous with glue.

Table 10–2. Standards for frequently used panel products

Product category	Applicable standard	Name of standard	Source
Plywood	PS 1–95	Voluntary product standard PS 1–95 Construction and industrial plywood	NIST 1995
	PS 2–92	Voluntary product standard PS 2–92 Performance standard for wood-based structural-use panels	NIST 1992
Oriented strandboard	PS 2–92	Voluntary product standard PS 2–92 Performance standard for wood-based structural-use panels	NIST 1992
Particleboard	ANSI A208.1–1993	Particleboard	NPA 1993
Hardboard	ANSI/AHA A135.4–1995	Basic hardboard	AHA 1995a
	ANSI/AHA A135.5–1995	Prefinished hardboard paneling	AHA 1995b
	ANSI/AHA A135.6–1990	Hardboard siding	AHA 1990
Insulation board	ASTM C208–94	Standard specification for cellulosic fiber insulating board	ASTM current edition
	ANSI/AHA A194.1–1985	Cellulosic fiberboard	AHA 1985
Medium-density fiberboard	ANSI A208.2–1994	Medium-density fiberboard (MDF)	NPA 1994

Product Standards

Product standards may be further classified as manufacturing method standards and laboratory test standards. Probably the best example of a manufacturing method standard is Voluntary Product Standard PS 1–95 for construction and industrial plywood (NIST 1995). This standard specifies such matters as what wood species and grades of veneer may be used, what repairs are permissible, and how repairs must be made. For panels produced according to prescriptive manufacturing requirements, a comparison of wood failure to adhesive failure in small test specimens of plywood is the performance test specified.

A good example of a laboratory test product standard is the American National Standard for mat-formed particleboard, ANSI A208.1 (NPA 1993). The American National Standards Institute (ANSI) product standards for both particleboard and MDF are sponsored by the Composite Panel Association (CPA) in Gaithersburg, Maryland. The CPA is the association resulting from the 1997 consolidation of the U.S.-based National Particleboard Association and the Canadian Particleboard Association. This standard states that in laboratory tests, specimens show certain minimally acceptable physical and mechanical properties, identified by numeric values. The test values give some indication of product quality, but the tests on small specimens were not specifically developed to correlate with performance of whole panels in specific end-uses.

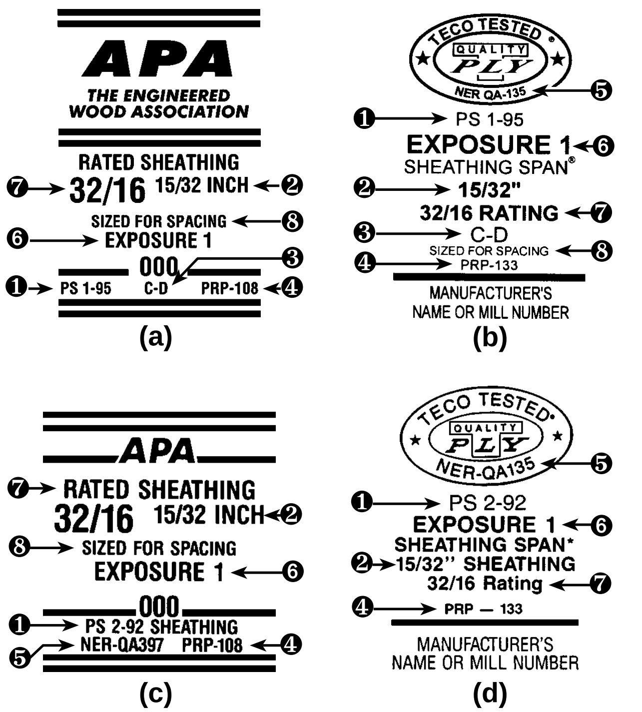
Performance Standards

Performance standards are written for panels in specific end-uses. These standards focus on panel performance in laboratory tests developed to indicate panel performance for particular end-uses. Federal legislation (Abourezk 1977) encourages the development of performance standards in preference to commodity-type standards. The Voluntary Standards and Accreditation Act of 1977 states that "a performance standard does not limit the manufacturer's freedom to choose any method of design or any form of construction that achieves the desired level of performance" (Abourezk 1977)

The APA–The Engineered Wood Association (formerly American Plywood Association) was the leading proponent of performance-type standards for panel products, and their early work formed the basis for the performance standards in existence today (O'Halloran 1979, 1980; APA 1981). Wood-based panels manufactured in conformance with performance standards (APA–The Engineered Wood Association 1995a, TECO 1991) are approved by the three major model codes by virtue of approval by the Council of American Building Officials through the issuance of a national evaluation report. These wood-based panels can be used for construction applications such as sheathing for roofs, subflooring, and walls.

Similarly, wood-based panels may be used in light-frame construction for many single-layer floor applications. Plywood, OSB, and COM-PLY, a proprietary product, are all span-rated for particular end uses.

Under PS 1–95 (NIST 1995), plywood panels intended for structural uses may be certified or rated using either prescriptive or performance-based criteria. Standard PS 2–92 (NIST 1992) is strictly performance based because it applies to all structural-use wood-based panels, including plywood, waferboard, and OSB; OSB is a second generation panel, with aligned fibers, that evolved from the original product called waferboard. The PS 2–92 standard is not a replacement for PS 1–95, which contains necessary veneer-grade and



❶ Product Standard that governs specifics of production for construction and industrial plywood

❷ Nominal panel thickness subject to acceptable tolerances

❸ Panel grade designation indicating minimum veneer grade used for panel face and back, or grade name based on panel use

❹ Performance-rated panel standard indicating structural-use panel test procedure recognized by National Evaluation Service (NES)

❺ NES report number from Council of American Building Officials (CABO)

❻ Exposure durability classification: Exposure 1 indicates interior panel bonded with exterior glue suitable for uses not permanently exposed to weather

❼ Span rating indicating maximum spacing of roof and floor supports for ordinary residential construction applications; 32/16 rating identifies a panel rated for use on roof supports spaced up to 813 mm (32 in.) o.c., or floor supports spaced up to 406 mm (16 in.) o.c.

❽ Sized for spacing denotes panels that have been sized to allow for spacing of panel edges during installation to reduce the possibility of buckling

Figure 10–4. Typical grade stamps for plywood and OSB.

adhesive-bond requirements as well as prescriptive lay-up provisions and includes many plywood grades not covered under PS 2–92.

A significant portion of the market for construction and industrial plywood is in residential construction. This situation has resulted in the development of performance standards for sheathing and single-layer subflooring or underlayment for residential construction. Plywood panels conforming to these performance standards for sheathing are marked with grade stamps such as those shown in Figure 10–4a,b. Structural flakeboards are usually marketed as conforming to a product standard for sheathing or single-layer subflooring or underlayment and are graded as a performance-rated product (PRP–108) similar to the grading for construction plywood. Voluntary Product Standard PS 2–92 is the performance standard for wood-based structural-use panels, which include such products as plywood, OSB, and waferboard. Panels conforming to these performance standards for sheathing are marked with grade stamps such as those shown in Figure 10–4c,d. As seen in Figure 10–4a,b, the grade stamps must show (1) conformance to plywood product standards, (2) nominal panel thickness, (3) grades of face and back veneers or grade name based on panel use, (4) performance-rated panel standard, (5) recognition as a quality assurance agency by the National Evaluation Service (NES), which is affiliated with the Council of American Building Officials, (6) exposure durability classification, (7) span rating, which refers to maximum allowable roof support spacing and maximum floor joist spacing, and (8) panel sizing for spacing.

Plywood

General Description

Plywood is a flat panel built up of sheets of veneer called plies, united under pressure by a bonding agent to create a panel with an adhesive bond between plies. Plywood can be made from either softwoods or hardwoods. It is always constructed with an odd number of layers with the grain direction of adjacent layers oriented perpendicular to one another. Since layers can consist of a single ply or of two or more plies laminated such that their grain is parallel, a panel can contain an odd or even number of plies but always an odd number of layers. The outside plies are called faces or face and back plies; the inner plies are called cores or centers; and the plies with grain perpendicular to that of the face and back are called crossbands. The core may be veneer, lumber, or particleboard, with the total panel thickness typically not less than 1.6 mm (1/16 in.) or more than 76 mm (3 in.). The plies may vary in number, thickness, species, and grade of wood. To distinguish the number of plies (individual sheets of veneer in a panel) from the number of layers (number of times the grain orientation changes), panels are sometimes described as three-ply, three-layer or four-ply, three-layer. The outer layers (face and back) and all odd-numbered layers (centers) generally have their grain direction oriented parallel to the length or long dimension of the panel. The grain of even-numbered layers (cores) is perpendicular to the length of the panel.

The alternation of grain direction in adjacent plies provides plywood panels with dimensional stability across their width. It also results in fairly similar axial strength and stiffness properties in perpendicular directions within the panel plane. The laminated construction distributes defects, markedly reduces splitting when the plywood is penetrated by fasteners (compared with splitting of solid wood), and improves resistance to checking.

Compared with solid wood, the chief advantages of plywood are that the properties along the length of the panel are more nearly equal to properties along the width, there is greater resistance to splitting, and the form permits many applications where large sheets are desirable. The use of plywood may result in improved utilization of wood. Plywood can cover large areas with a minimum amount of wood fiber because plywood that is thinner than sawn lumber can be used in some applications. The properties of plywood depend on the quality of the different layers of veneer, order of layer placement, adhesive used, and control of bonding conditions. The grade of the panel depends upon the quality of the veneers used, particularly of the face and back. The type of panel refers to the durability of the adhesive-to-wood bond and depends upon the adhesive-bonded joint, particularly its water resistance, and upon veneer grades used. Generally, face veneers with figured grain that are used in panels where appearance is important have numerous short, or otherwise deformed, wood fibers. These may significantly reduce strength and stiffness of the panels. On the other hand, face veneers and other plies may contain certain sizes and distributions of knots, splits, or growth characteristics that have no undesirable effects on strength properties for specific uses, such as sheathing for walls, roofs, or floors.

The plywood industry continues to develop new products. Hence, the reader should always refer directly to current specifications on plywood and its use for specific details.

Types of Plywood

Broadly speaking, two classes of plywood are available, covered by separate standards: (a) construction and industrial, and (b) hardwood and decorative. Construction and industrial plywood has traditionally been made from softwoods such as Douglas-fir, Southern Pine, white fir, larch, western hemlock, and redwood. However, the current standard lists a large number of hardwoods as qualifying for use. At the same time, the standard for hardwood and decorative plywood covers certain decorative softwood species for nonconstruction use.

Most construction and industrial plywood used in the United States is produced domestically, and U.S. manufacturers export some material. Generally speaking, the bulk of construction and industrial plywood is used where strength, stiffness, and construction convenience are more important than appearance. However, some grades of construction and industrial plywood are made with faces selected primarily for appearance and are used either with clear natural finishes or pigmented finishes.

Hardwood and decorative plywood is made of many different species, both in the United States and overseas. Well over half of all such panels used in the United States are imported. Hardwood plywood is normally used in such applications as decorative wall panels and for furniture and cabinet panels where appearance is more important than strength. Most of the production is intended for interior or protected uses, although a very small proportion is made with adhesives suitable for exterior service, such as in marine applications. A significant portion of all hardwood plywood is available completely finished.

The adhesives used in the manufacture of the two classes of plywood are quite different, but each type is selected to provide the necessary performance required by the appropriate specifications.

Construction and industrial plywood covered by Product Standard PS 1 is classified by exposure capability (type) and grade. The two exposure capabilities are exterior and interior. Exterior plywood is bonded with exterior adhesive, and veneers used in manufacture cannot be less than "C" grade as defined in PS 1. Interior-type plywood may be bonded with interior, intermediate, or exterior (waterproof) adhesive. "D" grade veneer is allowed as inner and back plies of certain interior-type plywoods. Adhesive bond performance requirements are specified in PS 1.

The four types of hardwood and decorative plywood in decreasing order of resistance to water are Technical (Exterior), Type I (Exterior), Type II (Interior), and Type III (Interior); adhesive bond requirements for these are specified in ANSI/HPVA–1–1994 (HPVA 1994).

Processing Considerations

After trees are felled and bucked to length, the logs are graded and sorted to make the most appropriate and efficient use of the wood fiber. For softwood plywood, in the past, logs graded as "peelers" were sent to veneer mills or plywood plants and "sawlogs" were shipped to lumber mills. Because of the dwindling availability of the clear, large-diameter peeler logs on which the plywood industry was founded, this practice has changed. Today, the higher grades of softwood peeler logs are sent to sawmills, and with few exceptions, plywood is made from low-grade sawlogs or peeler logs. This change came about because of the increasing demand for clear sawn lumber, and it has been made possible by innovations in veneer and plywood manufacturing and testing practices that ensure that panels are suitable for their intended use (McKay 1997).

Logs delivered to a veneer mill are sorted by grade and species, then debarked and crosscut into peeler blocks. Peeler blocks are often heated or conditioned by steaming or immersion in hot water prior to peeling, which makes them easier to peel, reduces veneer breakage, and results in smoother, higher quality veneer. The heated blocks are then conveyed to a veneer lathe. To maximize veneer yield, each block is gripped on the ends at the block's geometric center. While rotating at high speed, the block is fed against a stationary knife parallel to its length. Veneer is peeled from the block in a continuous, uniformly thin sheet, much like unwinding a roll of paper towels, but at a speed of up to 4.1 m/s (13.3 linear ft/s).

Depending on its intended use, veneer may range in thickness from 1.6 to 4.8 mm (1/16 to 3/16 in.) for softwood plywood and much thinner for hardwood and decorative plywood. After being peeled to a diameter from 127 to 51 mm (5 to 2 in.), the peeler core is ejected from the lathe. Peeler cores may be sawn into standard 38- by 89-mm (nominal 2- by 4-in.) lumber, used for fence posts, and landscape timbers, or chipped for use as pulp chips or fuel.

The continuous sheet of veneer is then transported by conveyor to a clipping station where it is clipped into usable widths and defects are removed. The wet veneer is then dried to an average moisture content that is compatible with the adhesive system being used to bond the panels. Since it is critical that veneer moisture content be low at the time adhesive is applied, each sheet is metered as it exits the dryer. Pieces that are too wet or dry are rerouted to be redried or reconditioned, respectively. Properly dried veneer is then sorted into one of as many as 15 to 20 different grades according to the size and number of knots and other natural and processing defects. Each grade has a specific use; some veneer requires special processing before it is assembled into plywood. After grading and/or processing, the veneer is taken to the lay-up area.

Adhesive is applied to veneers in the lay-up area by spray, curtain coating, roller coating, extrusion, and recently, foaming. Veneer is laid up into plywood by hand, machine, or a combination of both. Hand lay-up is the oldest method, and it is still the only practical way of making plywood for some applications. With this method, the face, back, and center veneers are hand-placed by workers called sheet turners. After being coated on both sides with adhesive, the alternating core plies are placed by hand or machine. The lay-up process is almost completely automated in newer plywood plants, although the narrow strips used for cores may still be placed manually. Before veneers are laid up, narrow strips are sometimes joined into full-width sheets with hot-melt adhesive-coated fiberglass thread so that they can be handled by machine. Also, veneers may be upgraded by punching out knots and other defects and replacing them with wood plugs or synthetic patches.

Once assembled, panels are conveyed from the lay-up area to the pressing area. Panels are first subjected to cold prepressing to flatten the veneers and transfer the adhesive to uncoated sheets; panels are then hot pressed. After hot pressing, panels are solid-piled or hot-stacked to ensure complete curing of the adhesive, then sawn to size. Panels are then graded with regard to the product standard under which they were manufactured. Knotholes and splits on the faces and backs of some panels may be repaired with wood plugs or with synthetic patches (by filling the holes and splits with what is essentially liquid plastic that quickly hardens). Those panels that do not meet the specification are downgraded or rejected. Panels needing further processing are sent to the finishing area where, depending on their intended use, they may be sanded to thickness, profiled with tongue and groove edges, surface textured, scarf- or finger-jointed, oiled and edge-sealed, or given other treatments. The panels are then ready for shipping (McKay 1997).

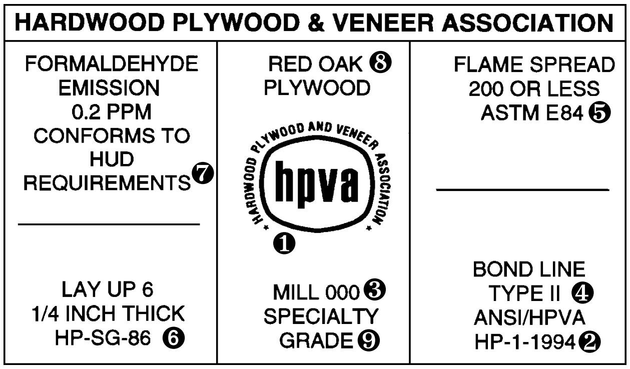
Specifications

The two general classes of plywood—(a) construction and industrial plywood and (b) hardwood and decorative plywood—are covered by separate standards. Construction and industrial plywood are covered by Product Standard PS 1–95 (NIST 1995), and hardwood and decorative plywood by American National Standard ANSI/HPVA–1–1994 (HPVA 1994). Each standard recognizes different exposure durability classifications, which are primarily based on moisture resistance of the adhesive and the grade of veneer used.

Model building codes in the United States stipulate that plywood used for structural applications like subflooring and sheathing must meet the requirements of certain U.S. Department of Commerce standards. Voluntary Product Standard PS 1–95 for construction and industrial plywood (NIST 1995) and Performance Standard PS 2–92 for wood-based structural-use panels (NIST 1992) spell out the ground rules for manufacturing plywood and establishing plywood or OSB properties, respectively. These standards have evolved over time from earlier documents (O'Halloran 1979, 1980; APA 1981) and represent a consensus opinion of the makers, sellers, and users of plywood products as well as other concerned parties. In addition, model building codes require that plywood manufacturers be inspected and their products certified for conformance to PS 1–95, PS 2–92, APA PRP–108, or TECO PRP–133 by qualified independent third-party agencies on a periodic unannounced basis.

With PS 1–95, as long as a plywood panel is manufactured using the veneer grades, adhesive, and construction established in the standard's prescriptive requirements, the panel is by definition acceptable. When plywood is assembled so that the proportion of wood with the grain perpendicular to the panel's face grain is greater than 33% or more than 70% of the panel's thickness, the plywood automatically meets the span rating. In panels with four or more plies, the combined thickness of the inner layers must equal 45% or more of the panel's thickness. Generally speaking, for panels of the same thickness and made with the face and back veneer of the same species, stiffness and strength increase as the thickness of the face and back veneers increases. All other things being equal, the stiffness and strength of plywood also increase as panel thickness increases.

All hardwood plywood represented as conforming to American National Standard ANSI/HPVA–1–1994 (HPVA 1994) is identified by one of two methods: by marking each panel with the Hardwood Plywood & Veneer Association (HPVA) plywood grade stamp (Fig. 10–5) or by including a written statement with this information with the order or shipment. The HPVA grade stamp shows (1) HPVA trademark, (2) standard that governs manufacture, (3) HPVA mill number, (4) plywood adhesive bond type, (5) flame spread index class, (6) description of lay-up, (7) formaldehyde emission characteristics, (8) face species, and (9) veneer grade of face.



Explanation of numbering

❶ HPVA trademark

❷ Standard governing manufacture

❸ HPVA mill number

❹ Plywood bondline type

❺ Flame spread index class as determined by testing in accordance with ASTM E84, standard test method for surface burning characteristics of building materials

❻ Lay-up description references structural attributes of wall panels as described in HPMA design guide HP-SG-86, Structural design Guide for Hardwood Plywood Wall Panels as published by the Hardwood Plywood & Veneer Association

❼ Formaldehyde emission characteristics to determine compliance with U.S. Department of Housing and Urban Development requirements for building product use in manufactured homes by testing in accordance with ASTM E1333, Standard Test Method for Determining Formaldehyde Levels From Wood Products Under Defined Test Conditions Using a Large Chamber

❽ Face species (face species designation is not required for wall panels when the surface is a decorative simulation such as that of a wood grain of another species or of a pattern)

❾ Veneer grade of face (grade of veneer of back is shown following grade of face for industrial panels)

Figure 10–5. Grade stamp for hardwood plywood conforming to ANSI/HPVA–1–1994.

The span-rating system for plywood was established to simplify plywood specification without resorting to specific structural engineering design. This system indicates performance without the need to refer to species group or panel thickness. It gives the allowable span when the face grain is placed across supports with a minimum of three supports.

If design calculations are desired, a design guide is provided by the APA–The Engineered Wood Association in *Plywood Design Specification* (PDS) and APA Technical Note N375B (APA–The Engineered Wood Association 1995a,b). The design guide contains tables of grade stamp references, section properties, and allowable stresses for plywood used in construction of buildings and similar structures.

Grades and Classification

Plywood is classified by both exposure durability class and grade. Exposure durability class refers to the ability of a panel to resist the damaging effects of exposure to the weather or moisture. Panel grades are either names that describe the intended use of the panel, such as underlayment or concrete form, or letters that identify the grades of the face and back veneers, such as A–B.

Veneers for plywood are visually graded according to the size, number, and location of natural and processing defects that affect their strength and appearance. Knots, decay, splits, insect holes, surface roughness, number of surface repairs, and other defects are considered. More surface repairs, such as elliptical (boat-shaped) wood patches and bigger knots are allowed in the lower veneer grades. Veneers are graded as N, A, B, C, C-Plugged, and D. N-grade or natural finish veneers are virtually blemish-free, and they contain only a few minor surface repairs. A and B veneers have solid surfaces with neatly made repairs and small, tight knots. Knotholes up to 25 mm (1 in.) in diameter are allowed in C veneers, whereas D veneers may have knotholes as large as 51 mm (2 in.) across. Because their appearance is usually of secondary importance, panels meant for sheathing and other structural uses are made mostly from C and D veneers. The N, A, and B veneers are reserved for panels where appearance is the primary consideration in such uses as exterior trim and soffits, interior paneling, doors, and cabinets.

Construction Plywood Exposure Durability Class

The exposure durability classifications for construction and industrial plywood specified in PS–1 are as follows: exterior, exposure 1, intermediate adhesive, exposure 2, and interior. Exterior plywood is bonded with exterior (waterproof) adhesive and is composed of C-grade or better veneers throughout. Exposure 1 plywood is bonded with exterior adhesives, but it may include D-grade veneers. Exposure 2 plywood is made with adhesive of intermediate resistance to moisture. Interior-type plywood may be bonded with interior, intermediate, or exterior (waterproof) adhesive. D-grade veneer is allowed on inner and back plies of certain interior-type grades.

The exposure durability classifications for hardwood and decorative plywood specified in ANSI/HPVA HP–1–1994 are as follow, in decreasing order of moisture resistance: technical (exterior), type I (exterior), type II (interior), and type III (interior). Hardwood and decorative plywood are not typically used in applications where structural performance is a prominent concern. Therefore, most of the remaining discussion of plywood performance will concern construction and industrial plywood.

Plywood Grades

There are many plywood grade names (Tables 10–3 and 10–4). In addition to the 30 or so generic names listed in PS 1–95, each agency that inspects plywood mills and certifies their products has coined its own trademarked grade names. For example, panels intended for use as single-layer

Table 10–3. Grade names for interior plywood grades[a]

Panel grade designation	Minimum face	Veneer back	Quality inner plies	Surface
N–N	N	N	C	S2S[b]
N–A	N	A	C	S2S
N–B	N	B	C	S2S
N–D	N	D	D	S2S
A–A	A	A	D	S2S
A–B	A	B	D	S2S
A–D	A	D	D	S2S
B–B	B	B	D	S2S
B–D	B	D	D	S2S
Underlayment	C plugged	D	C & D	Touch sanded
C–D plugged	C plugged	D	D	Touch sanded
Structural I C–D				Unsanded
Structural I C–D plugged, underlayment				Touch sanded
C–D	C	D	D	Unsanded
C–D with exterior adhesive	C	D	D	Unsanded

[a]NIST 1995.
[b]Sanded on two sides.

Table 10–4. Grade names for exterior plywood grades[a]

Panel grade designation	Minimum face	Veneer back	Quality inner plies	Surface
Marine, A–A, A–B. B–B, HDO, MDO				See regular grades
Special exterior, A–A, A–B, B–B, HDO, MDO				See regular grades
A–A	A	A	C	S2S[b]
A–B	A	B	C	S2S
A–C	A	C	C	S2S
B–B (concrete form)				
B–B	B	B	C	S2S
B–C	B	C	C	S2S
C–C plugged	C plugged	C	C	Touch sanded
C–C	C	C	C	Unsanded
A–A high-density overlay	A	A	C plugged	—
B–B high-density overlay	B	B	C plugged	—
B–B high-density concrete form overlay	B	B	C plugged	—
B–B medium-density overlay	B	B	C	—
Special overlays	C	C	C	—

[a]NIST 1995.
[b]Sanded on two sides.

flooring (combined subfloor and underlayment) made by TECO-certified manufacturers are called Floorspan, while those made by mills certified by the APA–The Engineered Wood Association are named Sturd-I-Floor. Although the trade names may be different, the minimum stiffness and strength properties of the panels are not. With the exception of custom-order panels, plywood is strictly a commodity product; panels of the same grade and thickness conforming to either PS 1–95 or PS 2–92 are interchangeable among manufacturers.

Span Rating and General Property Values

The more than 70 species of wood used for making softwood plywood (including some hardwoods) are classified into five groups according to their stiffness and strength (Table 10–5). The strongest woods are in Group 1; the weakest, in Group 5. Today, almost all plywood intended for structural use is marked with a two-number span rating (for example, 32/16) instead of a species group number (Fig. 10–4).

As with softwood lumber allowable design values, plywood span ratings were developed by breaking thousands of full-size panels of varying construction and thickness. The left-hand number of the rating represents the maximum

Table 10–5. Softwood plywood species groups by stiffness and strength[a]

Group 1	Group 2	Group 3	Group 4	Group 5
Apitong	Cedar, Port Orford	Alder, red	Aspen	Basswood
Beech, American	Cypress	Birch, paper	Bigtooth	Poplar
Birch	Douglas-fir[b]	Cedar, yellow	Quaking	Balsam
Sweet	Fir	Fir, subalpine	Cativo	
Yellow	Balsam	Hemlock, eastern	Cedar	
Douglas-fir[c]	California red	Maple, bigleaf	Incense	
Kapur	Grand	Pine	Western	
Keruing	Noble	Jack	Red	
Larch, western	Pacific silver	Lodgepole	Cottonwood	
Maple, sugar	White	Ponderosa	Eastern	
Pine	Hemlock, western	Spruce	Black	
Caribbean	Lauan	Redwood	(Western Poplar)	
Ocote	Almon	Spruce	Pine, eastern	
Pine, Southern	Bagtikan	Engelman	White, sugar	
Loblolly	Mayapis	White		
Longleaf	Red lauan			
Shortleaf	Tangile			
Slash	White lauan			
Tanoak	Maple, black			
	Mengkulang			
	Meranti, red			
	Mersawa			
	Pine			
	Pond			
	Red			
	Virginia			
	Western white			
	Spruce			
	Black			
	Red			
	Sitka			
	Sweetgum			
	Tamarack			
	Yellow poplar			

[a]From NIST 1995. Strongest species in Group 1; weakest in Group 5.
[b]Trees grown in Nevada, Utah, Colorado, Arizona, and New Mexico.
[c]Trees grown in Washington, Oregon, California, Idaho, Montana, Wyoming, and Canadian provinces of Alberta and British Columbia.

recommended on-center (OC) spacing for framing when the panel is used as roof sheathing; the right-hand number is the maximum recommended OC spacing for framing when the panel is used as subflooring. Panels intended for single-layer flooring (combined subfloor and underlayment) have only one span-rating number; for example, 24 OC. In all cases, the panels are meant to be installed with their length perpendicular to framing and across three or more supports. Again, panels of the same grade and span rating can be substituted for one another regardless of who made or certified them.

Table 10–6 provides approximate properties of sheathing-grade plywood. Plywood may be used under loading conditions that require the addition of stiffeners to prevent it from buckling. It may also be used in the form of cylinders or curved plates, which are beyond the scope of this handbook but are discussed in U.S. Department of Defense Bulletin ANC–18.

It is obvious from its construction that a strip of plywood cannot be as strong in tension, compression, or bending as a strip of solid wood of the same size. Those layers having

Table 10–6. General property values for sheathing-grade plywood[a]

Property	Value	ASTM test method[b] (where applicable)
Linear hygroscopic expansion (30%–90% RH)	0.15%	
Linear thermal expansion	6.1×10^{-6} cm/cm/°C (3.4×10^{-6} in/in/°F)	
Flexure		
Modulus of rupture	20.7–48.3 MPa (3,000–7,000 lb/in^2)	D3043
Modulus of elasticity	6.89–13.1 GPa (1–1.9 $\times 10^6$ lb/in^2)	
Tensile strength	l0.3–27.6 MPa (1,500–4,000 lb/in^2)	D3500
Compressive strength	20.7–34.5 MPa (3,000–5,000 lb/in^2)	D3501
Shear through thickness (edgewise shear)		
Shear strength	4.1–7.6 MPa (600–1,100 lb/in^2)	D2719
Shear modulus	0.47–0.761 GPa (68–110 $\times 10^3$ lb/in^2)	D3044
Shear in plane of plies (rolling shear)		D2718
Shear strength	1.7–2.1 MPa (250–300 lb/in^2)	
Shear modulus	0.14–0.21 GPa (20–30 $\times 10^3$ lb/in^2)	

[a]All mechanical properties are based on gross section properties of plywood panels, with stress applied parallel to grain direction of face plies where applicable. Note: Data are not to be used in developing allowable design values. Information on engineering design methods for plywood courtesy of APA–The Engineered Wood Association, Tacoma, WA.
[b]Standard methods of testing strength and elastic properties of structural panels are given in ASTM standards (see References).

their grain direction oriented at 90° to the direction of stress can contribute only a fraction of the strength contributed by the corresponding areas of a solid strip because they are stressed perpendicular to the grain. Strength properties in the length and width directions tend to be equalized in plywood because adjacent layers are oriented at an angle of 90° to each other.

Characteristics

Although plywood is an engineered wood product, it is also used as a component in other engineered wood products and systems in applications such as prefabricated I-joists, box beams, stressed-skin panels, and panelized roofs. Plywood has high strength-to-weight and strength-to-thickness ratios, and its stiffness and strength are more equal in width and length than are stiffness and strength of solid wood. Plywood also has excellent dimensional stability along its length and across its width. Minimal edge-swelling makes plywood perhaps the best choice for adhesive-bonded tongue-and-groove joints, even where some wetting is expected. Because the alternating grain direction of its layers significantly reduces splitting, plywood is an excellent choice for uses that call for fasteners to be placed very near the edges of a panel. In uses where internal knotholes and voids may pose a problem, such as in small pieces, plywood can be ordered with a solid core and face veneers.

Other Considerations

Plywood of thin, cross-laminated layers is very resistant to splitting. Therefore, nails and screws can be placed close together and close to the edges of panels. Of course, highly efficient, rigid joints can be obtained by bonding plywood to itself or to heavier wood members, such as those needed in prefabricated wood I-joists, box beams, and stressed-skin panels. Adhesive-bonded joints should not be designed to transmit load in tension primarily normal to the plane of the plywood sheet because of the rather low tensile strength of wood perpendicular to grain. Adhesive-bonded joints should be arranged to transmit loads through shear. It must be recognized that shear strength across the grain of wood (often called rolling shear strength because of the tendency to roll the wood fibers) is only 20% to 30% of that parallel to the grain. Thus, sufficient area must be provided between plywood and flange members of box beams and between plywood and stringers of stressed-skin panels to avoid perpendicular-to-grain shearing failure in the face veneer, in the crossband veneer next to the face veneer, or in the wood member. Various details of design are given in Chapter 11.

Specialty Panels

Some plywood panels are designed for special uses, including marine decorative underlayment and concrete form and special exterior applications. The treating of plywood with preservatives and fire retardants is done by manufacturers outside of the plywood industry. Plywood is easily pressure-treated with waterborne preservatives and fire retardants, and treated plywood is readily available for use where such protection is needed.

Particle and Fiber Composites

Many wood-based composite materials have become popular. These composites are usually available in panel form and are widely used in housing and furniture. Conventional composites are typically made with a heat-curing adhesive that holds the wood fiber components together. The physical and mechanical properties of wood-based fiber and particle panel materials are determined by standard ASTM test methods.

General Processing Considerations

All the products in the family of particle and fiber composite materials are processed in similar ways. Raw material for OSB, waferboard, and fiberboard is obtained by flaking or chipping roundwood. For fiberboard, chips are reduced to wood fiber using refiners that usually use steam to soften the wood. The comminuted wood is then dried, adhesive is applied, and a mat of wood particles, fibers, or strands is formed; the mat is then pressed in a platen-type press under heat and pressure until the adhesive is cured. The bonded product is allowed to cool and is further processed into specified width, length, and surface qualities.

Oriented Strandboard

Oriented strandboard is an engineered structural-use panel manufactured from thin wood strands bonded together with waterproof resin under heat and pressure, and it is used extensively for roof, wall, and floor sheathing in residential and commercial construction. Orientation of wood strands with a typical aspect ratio (that is, strand length divided by width) of at least 3 can produce a panel product with greater bending strength and stiffness in the oriented or aligned direction.

Raw Materials

The raw material for the original waferboard product, which was made from square wafers, was aspen. As this industry expanded and OSB became the predominant product manufactured, other species such as Southern Pine, white birch, red maple, sweetgum, and yellow-poplar were found to be suitable raw materials as well. Small amounts of some other hardwoods can also be used for OSB.

Manufacturing Process

In the general manufacturing process for OSB, debarked logs are often heated in soaking ponds, then sliced into thin wood elements. The strands are dried, blended with resin and wax, and formed into thick, loosely consolidated mats that are pressed under heat and pressure into large panels.
Figure 10–6 shows an OSB manufacturing process. Oriented strandboard is made from long, narrow strands, with the strands of each layer aligned parallel to one another but perpendicular to strands in adjacent layers, like the cross-laminated veneers of plywood. It is this perpendicular orientation of different layers of aligned strands that gives OSB its unique characteristics and allows it to be engineered to suit different uses.

Stranding Process

Typically, logs are debarked and then sent to a soaking pond or directly to the stranding process. Long log disk or ring stranders are commonly used to produce wood strands typically measuring 114 to 152 mm (4.5 to 6 in.) long, 12.7 mm (0.5 in.) wide, and 0.6 to 0.7 mm (0.023 to 0.027 in.) thick.

Drying Process

Green strands are stored in wet bins and then dried in a traditional triple-pass dryer, a single-pass dryer, a combination triple-pass/single-pass dryer, or a three-section conveyor dryer. A relatively recent development is a continuous chain dryer, in which the strands are laid on a chain mat that is mated with an upper chain mat and the strands are held in place as they move through the dryer. The introduction of new drying techniques allows the use of longer strands, reduces surface inactivation of strands, and lowers dryer outfeed temperatures. Dried strands are screened and sent to dry bins.

Adhesive Application or Blending

The blending of strands with adhesive and wax is a highly controlled operation, with separate rotating blenders used for face and core strands. Typically, different resin formulations are used for face and core layers. Face resins may be liquid or powdered phenolics, whereas core resins may be phenolics or isocyanates. Several different resin application systems are used; spinning disk resin applicators are frequently used.

Mat Formation

Mat formers take on a number of configurations, ranging from electrostatic equipment to mechanical devices containing spinning disks to align strands along the panel's length and star-type cross-orienters to position strands across the panel's width. All formers use the long and narrow characteristic of the strand to place it between the spinning disks or troughs before it is ejected onto a moving screen or conveyor belt below the forming heads. Oriented layers of strands within the mat—face, core, face, for example—are dropped sequentially, each by a different forming head. Modern mat formers either use wire screens laid over a moving conveyor belt to carry the mat into the press or screenless systems in which the mat lies directly on the conveyor belt.

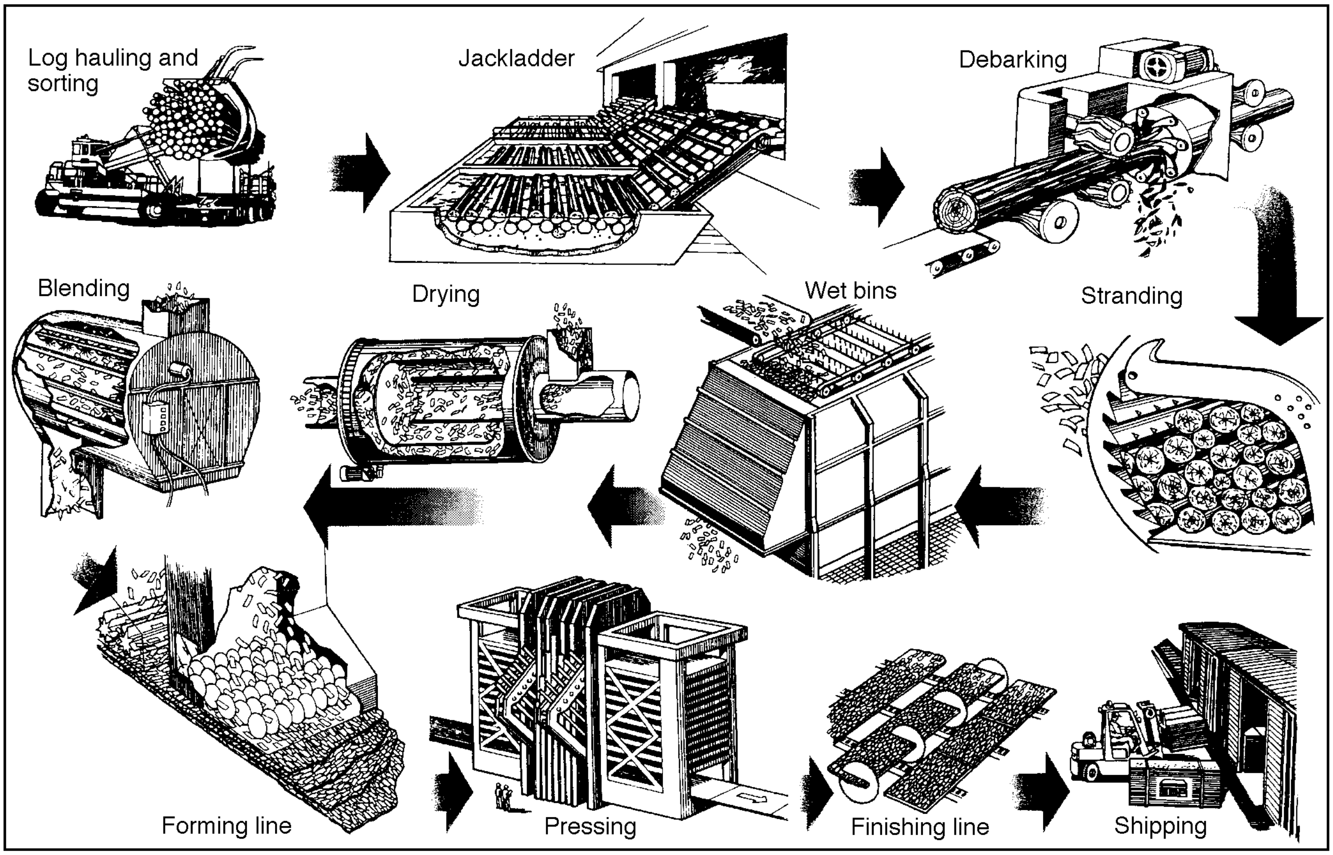


Figure 10–6. Schematic of OSB manufacturing process. (Courtesy of Structural Board Association, Willowdale, Ontario, Canada.)

Hot Pressing

In hot pressing, the loose layered mat of oriented strands is compressed under heat and pressure to cure the resin. As many as sixteen 3.7- by 7.3-m (12- by 24-ft) panels may be formed simultaneously in a multiple-opening press. A more recent development is the continuous press for OSB. The press compacts and consolidates the oriented and layered mat of strands and heats it to 177°C to 204°C (350°F to 400°F) to cure the resin in 3 to 5 min.

Design Capacities and Panel Certification

Design capacities of performance-rated products, which include OSB and waferboard, can be determined by using procedures outlined in Technical Note N375B (APA–The Engineered Wood Association 1995a). In this reference, allowable design strength and stiffness properties, as well as nominal thickness and section properties, are specified based on the span rating of the panel. Additional adjustment factors based on panel grade and construction are also provided. Table 10–7 provides general property values for sheathing-grade OSB.

Under PS 2–92, a manufacturer is required to enter into an agreement with an accredited testing agency to demonstrate that its panels conform with the requirements of the chosen standard. The manufacturer must also maintain an in-plant quality control program in which panel properties are regularly checked, backed by an independent third-party-administered quality assurance program. The third-party agency must visit the mill on a regular unannounced basis. The agency must confirm that the in-plant quality control program is being maintained and that panels meet the minimum requirements of the standard. Today, OSB manufactured to standard PS 2–92 is quality-certified by the following organizations: APA–The Engineered Wood Association, Professional Services and Industries, Inc., Pittsburgh Testing Laboratories, and PFS/TECO Corporations. Examples of grade stamps for performance-rated panels are shown in Figure 10–4c,d.

Particleboard

The wood particleboard industry grew out of a need to dispose of large quantities of sawdust, planer shavings, and to a lesser extent, the use of mill residues and other relatively homogeneous waste materials produced by other wood industries. Simply put, particleboard is produced by mechanically reducing the material into small particles, applying adhesive to the particles, and consolidating a loose mat of

Table 10–7. General property values for sheathing-grade OSB[a]

Property	Value	ASTM test method[b] (where applicable)
Linear hygroscopic expansion (30%–90% RH)	0.15%	
Linear thermal expansion	6.1×10^{-6} cm/cm/°C (3.4×10^{-6} in/in/°F)	
Flexure		
Modulus of rupture	20.7–27.6 MPa (3,000–4,000 lb/in^2)	D3043
Modulus of elasticity	4.83–8.27 GPa (700–1,200 $\times 10^3$ lb/in^2)	
Tensile strength	6.9–l0.3 MPa (1,000–1,500 lb/in^2)	D3500
Compressive strength	10.3–17.2 MPa (1,500–2,500 lb/in^2)	D3501
Shear through thickness (edgewise shear)		
Shear strength	6.9–10.3 MPa (1,000–1,500 lb/in^2)	D2719
Shear modulus	1.24–2.00 GPa (180–290 $\times 10^3$ lb/in^2)	D3044
Shear in plane of plies (rolling shear)		D2718
Shear strength	1.38–2.1 MPa (200–300 lb/in^2)	
Shear modulus	0.14–0.34 GPa (20–50 $\times 10^3$ lb/in^2)	

[a]All mechanical properties are based on gross section properties of OSB panels, with stress applied parallel to panel major axis where applicable. Note: Data are not to be used in developing allowable design values. Information courtesy of APA–The Engineered Wood Association, Tacoma, WA.
[b]Standard methods of testing strength and elastic properties of structural panels are given in ASTM standards (see References).

the particles with heat and pressure into a panel product (Fig. 10–7). All particleboard is currently made using a dry process, where air or mechanical formers are used to distribute the particles prior to pressing.

Particleboard is typically made in three layers. The faces of the board consists of fine wood particles, and the core is made of the coarser material. Producing a panel this way improves utilization of the material and the smooth face presents a better surface for laminating, overlaying, painting, or veneering. Particleboard is also readily made from a variety of agricultural residues. Low-density insulating or sound-absorbing particleboard can be made from kenaf core or jute stick. Low-, medium-, and high-density panels can be produced with cereal straw, which has begun to be used in North America. Rice husks are commercially manufactured into medium- and high-density products in the Middle East.

All other things being equal, reducing lignocellulosic materials to particles requires less energy than reducing the same material into fibers. However, particleboard is generally not as strong as fiberboard because the fibrous nature of lignocellulosics is not exploited as well. Particleboard is used for furniture cores, where it is typically overlaid with other materials for decorative purposes. Particleboard can be used in flooring systems, in manufactured houses, for stair treads, and as underlayment. Thin panels can be used as a paneling substrate. Since most applications are interior,

Figure 10–7. Particles, which are sometimes produced by hammermilling, are used to produce composites such as particleboard.

particleboard is usually bonded with a UF resin, although PF and MF resins are sometimes used for applications requiring more moisture resistance. The various steps involved in particleboard manufacturing are described in the following text.

Particle Preparation

Standard particleboard plants based on particulate material use combinations of hogs, chippers, hammermills, ring flakers, ring mills, and attrition mills. To obtain particleboards with good strength, smooth surfaces, and equal swelling, manufacturers ideally use a homogeneous material with a high degree of slenderness (long, thin particles), no oversize particles, no splinters, and no dust. Depending on the manufacturing process, the specifications for the ideal particle size are different. For a graduated board, wider tolerances are acceptable. For a three-layer board, the core particles should be longer and surface particles shorter, thinner, and smaller. For a five-layer or multi-layer board, the furnish for the intermediate layer between surface and core should have long and thin particles for building a good carrier for the fine surface and to give the boards high bending strength and stiffness.

Particle Classification and Conveying

Very small particles increase furnish surface area and thus increase resin requirements. Oversized particles can adversely affect the quality of the final product because of internal flaws in the particles. While some particles are classified through the use of air streams, screen classification methods are the most common. In screen classification, the particles are fed over a vibrating flat screen or a series of screens. The screens may be wire cloth, plates with holes or slots, or plates set on edge.

The two basic methods of conveying particles are by mechanical means and by air. The choice of conveying method depends upon the size of the particles. In air conveying, care should be taken that the material does not pass through many fans, which reduces the size of the particles. In some types of flakes, damp conditions are maintained to reduce break-up of particles during conveying.

Particle Drying

The furnish drying operation is a critical step in the processing of composite products. The raw materials for these products do not usually arrive at the plant at a low enough moisture content for immediate use. Furnish that arrives at the plant can range from 10% to 200% moisture content. For use with liquid resins, for example, the furnish must be reduced to about 2% to 7% moisture content.

The moisture content of particles is critical during hot-pressing operations. Thus, it is essential to carefully select proper dryers and control equipment. The moisture content of the material depends on whether resin is to be added dry or in the form of a solution or emulsion. The moisture content of materials leaving the dryers is usually in the range of 4% to 8%. The main methods used to dry particles are rotary, disk, and suspension drying.

A triple-pass rotary dryer consists of a large horizontal rotating drum that is heated by either steam or direct heat. Operating temperatures depend on the moisture content of the incoming furnish. The drum is set at a slight angle, and material is fed into the high end and discharged at the low end. A series of flights forces the furnish to flow from one end to the other three times before being discharged. The rotary movement of the drum moves the material from input to output.

Addition of Resins and Wax

Frequently used resins for particleboard include urea-formaldehyde and, to a much lesser extent, phenol-formaldehyde, melamine-formaldehyde, and isocyanates. The type and amount of resin used for particleboard depend on the type of product desired. Based on the weight of dry resin solids and ovendry weight of the particles, the resin content can range between 4% and 10%, but usually ranges between 6% and 9% for UF resins. The resin content of the outer face layers is usually slightly higher than that of the core layer. Urea-formaldehyde resin is usually introduced in water solutions containing about 50% to 65% solids. Besides resin, paraffin or microcrystalline wax emulsion is added to improve short-term moisture resistance. The amount of wax ranges from 0.3% to 1% based on the ovendry weight of the particles.

Mat Formation

After the particles have been prepared, they must be laid into an even and consistent mat to be pressed into a panel. This is typically accomplished in a batch mode or by continuous formation. The batch system employs a caul or tray on which a deckle frame is placed. The mat is formed by the back-and-forth movement of the tray or hopper feeder. The mat is usually cold pressed to reduce mat thickness prior to hot pressing. The production of three-layer boards requires three or more forming stations. The two outer layers consist of particles that differ in geometry from those in the core. The resin content of the outer layers is usually higher (about 8% to 15%) than that of the core (about 4% to 8%).

In continuous mat-forming systems, the particles are distributed in one or several layers on traveling cauls or on a moving belt. Mat thickness is controlled volumetrically. The two outer face layers usually consist of particles that differ in geometry from those in the core. Continuous-formed mats are often pre-pressed, with either a single-opening platen or a continuous press. Pre-pressing reduces mat height and helps to consolidate the mat for pressing.

Hot Pressing

After pre-pressing, the mats are hot-pressed into panels. Presses can be divided into platen and continuous types. Further development in the industry has made possible the construction of presses for producing increasingly larger panel sizes in both single- and multi-opening presses. Both of these types of presses can be as wide as 3.7 m (12 ft). Multi-opening presses can be as long as 10 m (33 ft) and single-opening presses, up to 30.5 m (100 ft) long. Hot-press temperatures for UF resins usually range from 140°C to 165°C (284°F to 325°F). Pressure depends on a number of factors, but it is usually in the range of 1.37 to 3.43 MPa (199 to 498 lb/in^2) for medium-density boards. Upon entering the hot press, mats usually have a moisture content of 8% to 12%, but this is reduced to about 5% to 9% during pressing.

Alternatively, some particleboards are made by the extrusion process. In this system, formation and pressing occur in one operation. The particles are forced into a long, heated die (made of two sets of platens) by means of reciprocating pistons. The board is extruded between the platens. The particles are oriented in a plane perpendicular to the plane of the board, resulting in properties that differ from those obtained with flat pressing.

Finishing

After pressing, the board is trimmed to obtain the desired length and width and to square the edges. Trim losses usually amount to 0.5% to 8%, depending on the size of the board, the process employed, and the control exercised. Trimmers usually consist of saws with tungsten carbide tips. After trimming, the boards are sanded or planed prior to packaging and shipping. Particleboards may also be veneered or overlaid with other materials to provide a decorative surface, or they may be finished with lacquer or paint. Treatments with fire-resistant chemicals are also available.

Properties

Tables 10–8 and 10–9 show requirements for grades of particleboard and particleboard flooring products, as specified by the American National Standard for Particleboard A208.1 (NPA 1993). This standard is typically updated at least every 5 years. Today, approximately 85% of interior-type particleboard is used as core stock for a wide variety of furniture and cabinet applications. Floor underlayment and manufactured home decking represent particleboard construction products and approximately 10% of the market. Low-density panels produced in thicknesses >27 mm (>1-1/16 in.) are used for solid-core doors.

Particleboard Grade Marks and Product Certification

Particleboard that has been grade marked ensures that the product has been periodically tested for compliance with voluntary industry product performance standards. These inspection or certification programs also generally require that the quality control system of a production plant meets strict criteria. Particleboard panels conforming to these product performance standards are marked with grade stamps such as those shown in Figure 10–8.

Fiberboard

The term fiberboard includes hardboard, medium-density fiberboard (MDF), and insulation board. Several things differentiate fiberboard from particleboard, most notably the physical configuration of the comminuted material (Fig. 10–9). Because wood is fibrous by nature, fiberboard exploits the inherent strength of wood to a greater extent than does particleboard.

To make fibers for composites, bonds between the wood fibers must be broken. In its simplest form, this is

Table 10–8. Particleboard grade requirements[a,b,c]

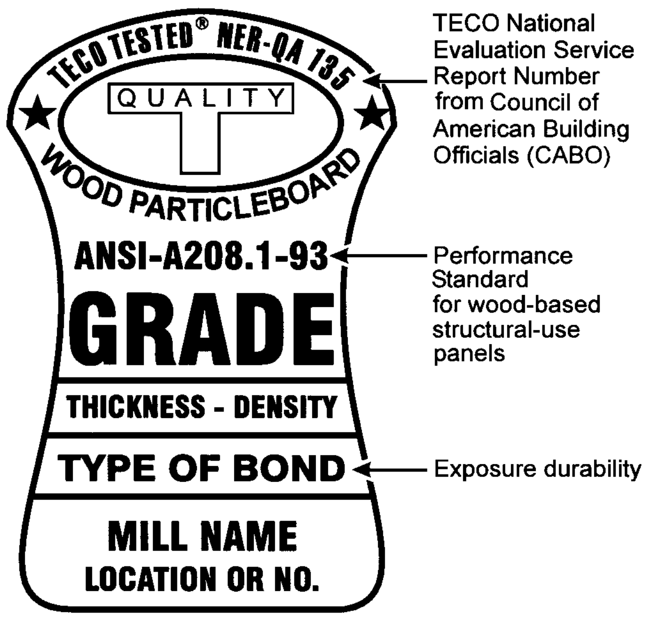
Grade[d]	MOR (MPa)	MOE (MPa)	Internal bond (MPa)	Hardness (N)	Linear expansion max avg (%)	Screw-holding (N) Face	Screw-holding (N) Edge	Formaldehyde maximum emission (ppm)
H–1	16.5	2,400	0.90	2,225	NS	1,800	1,325	0.30
H–2	20.5	2,400	0.90	4,450	NS	1,900	1,550	0.30
H–3	23.5	2,750	1.00	6,675	NS	2,000	1,550	0.30
M–1	11.0	1,725	0.40	2,225	0.35	NS	NS	0.30
M–S	12.5	1,900	0.40	2,225	0.35	900	800	0.30
M–2	14.5	2,225	0.45	2,225	0.35	1,000	900	0.30
M–3	16.5	2,750	0.55	2,225	0.35	1,100	1,000	0.30
LD–1	3.0	550	0.10	NS	0.35	400	NS	0.30
LD–2	5.0	1,025	0.15	NS	0.35	550	NS	0.30

[a]From NPA (1993). Particleboard made with phenol-formaldehyde-based resins does not emit significant quantities of formaldehyde. Therefore, such products and other particleboard products made with resin without formaldehyde are not subject to formaldehyde emission conformance testing.
[b]Panels designated as "exterior adhesive" must maintain 50% MOR after ASTM D1037 accelerated aging.
[c]MOR = modulus of rupture; MOE = modulus of elasticity. NS = not specified. 1 MPa = 145 lb/in^2; 1 N = 0.22 lb.
[d]H = density > 800 kg/m^3 (> 50 lb/ft^3), M = density 640 to 800 kg/m^3 (40 to 50 lb/ft^3). LD = density < 640 kg/m^3 (< 40 lb/ft^3). Grade M–S refers to medium density; "special" grade added to standard after grades M–1, M–2, and M–3. Grade M–S falls between M–1 and M–2 in physical properties.

Table 10–9. Particleboard flooring product grade requirements[a]

Grade[b]	MOR (MPa)	MOE (MPa)	Internal bond (MPa)	Hardness (N)	Linear expansion max avg (%)	Formaldehyde maximum emission (ppm)
PBU	11.0	1,725	0.40	2,225	0.35	0.20
D–2	16.5	2,750	0.55	2,225	0.30	0.20
D–3	19.5	3,100	0.55	2,225	0.30	0.20

[a]From NPA (1993). Particleboard made with phenol-formaldehyde-based resins does not emit significant quantities of formaldehyde. Therefore, such products and other particleboard products made with resin without formaldehyde are not subject to formaldehyde emission conformance testing. Grades listed here shall also comply with appropriate requirements listed in section 3. Panels designated as "exterior adhesive" must maintain 50% MOR after ASTM D1037 accelerated aging (3.3.3).
[b]PBU = underlayment; D = manufactured home decking.



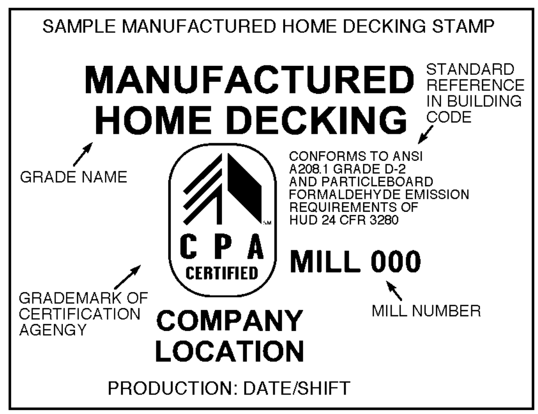


Figure 10–8. Examples of grade stamps for particleboard.

accomplished by attrition milling. Attrition milling is an age-old concept whereby material is fed between two disks, one rotating and the other stationary. As the material is forced through the preset gap between the disks, it is sheared, cut, and abraded into fibers and fiber bundles. Grain has been ground in this way for centuries.

Attrition milling, or refining as it is commonly called, can be augmented by water soaking, steam cooking, or chemical treatments. Steaming the lignocellulosic weakens the lignin bonds between the cellulosic fibers. As a result, the fibers are more readily separated and usually are less damaged than fibers processed by dry processing methods. Chemical treatments, usually alkali, are also used to weaken the lignin bonds. All of these treatments help increase fiber quality and reduce energy requirements, but they may reduce yield as well. Refiners are available with single- or double-rotating disks, as well as steam-pressurized and unpressurized configurations. For MDF, steam-pressurized refining is typical.

Fiberboard is normally classified by density and can be made by either dry or wet processes (Fig. 10–2). Dry processes are applicable to boards with high density (hardboard) and medium density (MDF). Wet processes are applicable to both high-density hardboard and low-density insulation board. The following subsections briefly describe the manufacturing of high- and medium-density dry-process fiberboard, wet-process hardboard, and wet-process low-density insulation board. Suchsland and Woodson (1986) and Maloney (1993) provide more detailed information.

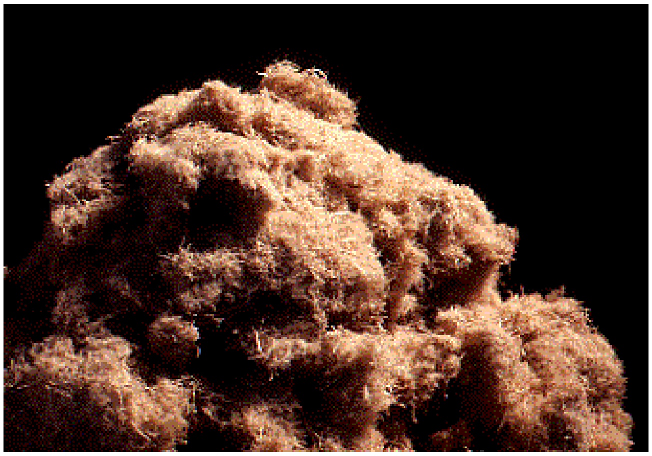


Figure 10–9. Fibers can be made from many lignocellulosics and form the raw materials for many composites, most notably fiberboard. Fibers are typically produced by the refining process.

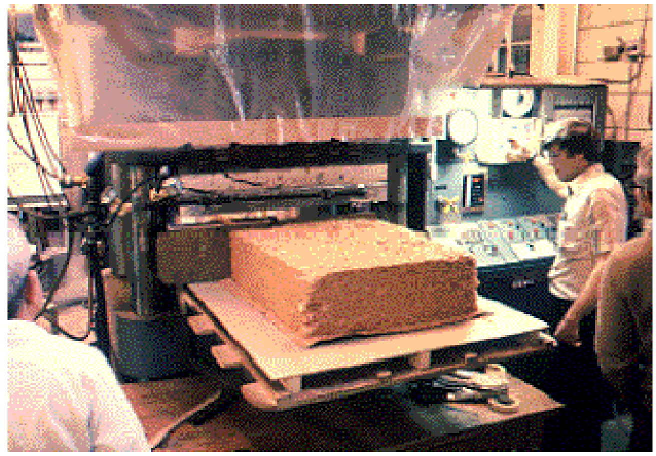


Figure 10–11. Air-laid mat about to enter a laboratory press.

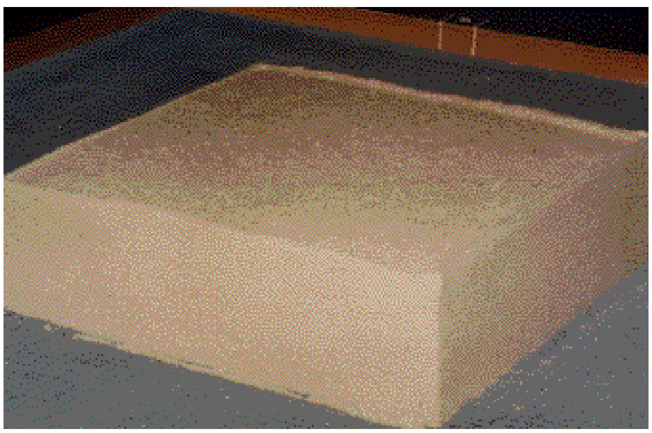


Figure 10–10. Laboratory-produced air-laid mat before pressing. Approximate dimensions are 686 by 686 by 152 mm (27 by 27 by 6 in.) thick. Resin was applied to fibers before mat production. This mat will be made into a high-density fiberboard approximately 3 mm (0.12 in.) thick.

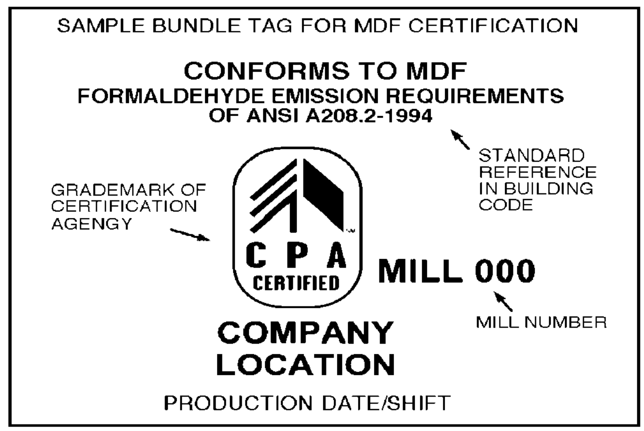


Figure 10–12. Example of MDF formaldehyde emissions certification tag.

Dry-Process Fiberboard

Dry-process fiberboard is made in a similar fashion to particleboard. Resin (UF, PF) and other additives may be applied to the fibers by spraying in short-retention blenders or introduced as the wet fibers are fed from the refiner into a blowline dryer. Alternatively, some fiberboard plants add the resin in the refiner. The adhesive-coated fibers are then air-laid into a mat for subsequent pressing, much the same as mat formation for particleboard.

Pressing procedures for dry-process fiberboard differ somewhat from particleboard procedures. After the fiber mat is formed (Fig. 10–10), it is typically pre-pressed in a band press. The densified mat is then trimmed by disk cutters and transferred to caul plates for the hardboard pressing operation; for MDF, the trimmed mat is transferred directly to the press (Fig. 10–11). All dry-formed boards are pressed in multi-opening presses at approximately 140°C to 165°C (284°F to 329°F) for UF-bonded products and 190°C (410°F) for PF-bonded products. Continuous pressing using large, high-pressure band presses is also gaining in popularity. Board density is a basic property and an indicator of board quality. Since density is greatly influenced by moisture content, this is constantly monitored by moisture sensors using infrared light. An example of an MDF formaldehyde emissions certification tag is shown in Figure 10–12.

Wet-Process Hardboard

Wet-process hardboards differ from dry-process fiberboards in several significant ways. First, water is used as the distribution medium for forming the fibers into a mat. As such, this technology is really an extension of paper manufacturing technology. Secondly, some wet-process boards are made without additional binders. If the lignocellulosic contains sufficient lignin and if lignin is retained during the refining operation, lignin can serve as the binder. Under heat and pressure, lignin will flow and act as a thermosetting adhesive, enhancing the naturally occurring hydrogen bonds.

Refining is an important step for developing strength in wet-process hardboards. The refining operation must also yield a fiber of high "freeness;" that is, it must be easy to remove water from the fibrous mat. The mat is typically formed on a Fourdrinier wire, like papermaking, or on cylinder formers. The wet process employs a continuously traveling mesh screen, onto which the soupy pulp flows rapidly and smoothly. Water is drawn off through the screen and then through a series of press rolls, which use a wringing action to remove additional water.

Wet-process hardboards are pressed in multi-opening presses heated by steam. The press cycle consists of three phases and lasts 6 to 15 min. The first phase is conducted at high pressure, and it removes most of the water while bringing the board to the desired thickness. The primary purpose of the second phase is to remove water vapor. The final phase is relatively short and results in the final cure. A maximum pressure of about 5 MPa (725 lb/in^2) is used. Heat is essential during pressing to induce fiber-to-fiber bond. A high temperature of up to 210°C (410°F) is used to increase production by causing faster evaporation of the water. Lack of sufficient moisture removal during pressing adversely affects strength and may result in "springback" or blistering.

Post-Treatment of Wet- and Dry-Process Hardboard

Several treatments are used to increase the dimensional stability and mechanical performance of hardboard. Heat treatment, tempering, and humidification may be done singularly or in conjunction with one another.

Heat treatment—exposure of pressed fiberboard to dry heat—improves dimensional stability and mechanical properties, reduces water adsorption, and improves interfiber bonding.

Tempering is the heat treatment of pressed boards, preceded by the addition of oil. Tempering improves board surface hardness and is sometimes done on various types of wet-formed hardboards. It also improves resistance to abrasion, scratching, scarring, and water. The most common oils used include linseed oil, tung oil, and tall oil.

Humidification is the addition of water to bring the board moisture content into equilibrium with the air. Initially, a pressed board has almost no moisture content. When the board is exposed to air, it expands linearly by taking on 3% to 7% moisture. Continuous or progressive humidifiers are commonly used for this purpose. Air of high humidity is forced through the stacks where it provides water vapor to the boards. The entire process is controlled by a dry-bulb-wet-bulb controller. Another method involves spraying water on the back side of the board.

Insulation Board

Insulation boards are low-density, wet-laid panel products used for insulation, sound deadening, carpet underlayment, and similar applications. In the manufacture of insulation board, the need for refining and screening is a function of the raw material available, the equipment used, and the desired end-product. Insulation boards typically do not use a binder, and they rely on hydrogen bonds to hold the board components together. Sizing agents are usually added to the furnish (about 1%) to provide the finished board with a modest degree of water resistance and dimensional stability. Sizing agents include rosin, starch, paraffin, cumarone, resin, asphalt, and asphalt emulsions.

Like the manufacture of wet-process hardboard, insulation board manufacture is a modification of papermaking. A thick fibrous sheet is made from a low-consistency pulp suspension in a process known as wet felting. Felting can be accomplished through use of a deckle box, Fourdrinier screen, or cylinder screen. A deckle box is a bottomless frame that is placed over a screen. A measured amount of stock is put in the box to form one sheet; vacuum is then applied to remove most of the water. The use of Fourdrinier screen for felting is similar to that for papermaking, except that line speeds are reduced to 1.5 to 15 m/min (5 to 49 ft/min).

Insulation board is usually cold-pressed to remove most of the free water after the mat is formed. The wet mats are then dried to the final moisture content. Dryers may be a continuous tunnel or a multi-deck arrangement. The board is generally dried in stages at temperatures ranging from 120°C to 190°C (248°F to 374°F). Typically, about 2 to 4 h are required to reduce moisture content to about 1% to 3%.

After drying, some boards are treated for various applications. Boards may be given tongue-and-groove or shiplap edges or can be grooved to produce a plank effect. Other boards are laminated by means of asphalt to produce roof insulation.

Properties and Applications

Medium-Density Fiberboard—Minimum property requirements, as specified by the American National Standard for MDF, A208.2 (NPA 1994) are given in Table 10–10. This standard is typically updated every 5 years or less. The furniture industry is by far the dominant MDF market. Medium-density fiberboard is frequently used in place of solid wood, plywood, and particleboard in many furniture applications. It is also used for interior door skins, mouldings, and interior trim components (Youngquist and others 1997).

Hardboard—Table 10–11 provides basic hardboard physical properties (ANSI/AHA A135.4–1995 (AHA 1995a)) for selected products. The uses for hardboard can generally be grouped as construction, furniture and furnishings, cabinet and store work, appliances, and automotive and rolling stock. Typical hardboard products are prefinished paneling (ANSI/AHA A135.5–1995 (AHA 1995b)), house siding (ANSI/AHA A135.6–1990 (AHA 1990)), floor underlayment, and concrete form board. Table 10–12 shows physical

Table 10–10. Medium-density fiberboard (MDF) property requirements[a]

Product class[b]	Nominal thickness (mm)	MOR (MPa)	MOE (MPa)	Internal bond (MPa)	Screw-holding (N)		Formaldehyde emission[c] (ppm)
					Face	Edge	
Interior MDF							
HD		34.5	3,450	0.75	1,555	1,335	0.30
MD	≤21	24.0	2,400	0.60	1,445	1,110	0.30
	>21	24.0	2,400	0.55	1,335	1,000	0.30
LD		14.0	1,400	0.30	780	670	0.30
Exterior MDF							
MD–Exterior	≤21	34.5	3,450	0.90	1,445	1,110	0.30
adhesive	>21	31.0	3,100	0.70	1,335	1,000	0.30

[a]From NPA (1994). Metric property values shall be primary in determining product performance requirements.
[b]MD–Exterior adhesive panels shall maintain at least 50% of listed MOR after ASTM D1037–1991, accelerated aging (3.3.4). HD = density > 800 kg/m^3 (> 50 lb/ft^3), MD = density 640 to 800 kg/m^3 (40 to 50 lb/ft^3), LD = density < 640 kg/m^3 (< 40 lb/ft^3).
[c]Maximum emission when tested in accordance with ASTM E1333–1990, Standard test method for determining formaldehyde levels from wood products under defined test conditions using a larger chamber (ASTM).

Table 10–11. Hardboard physical property requirements[a]

Product class	Normal thickness (mm)	Water resistance (max avg/panel)		MOR (min avg/panel) (MPa)	Tensile strength (min avg/panel) (MPa)	
		Water absorption based on weight (%)	Thickness swelling (%)		Parallel to surface	Perpendicular to surface
Tempered	2.1	30	25	41.4	20.7	0.90
	2.5	25	20	41.4	20.7	0.90
	3.2	25	20	41.4	20.7	0.90
	4.8	25	20	41.4	20.7	0.90
	6.4	20	15	41.4	20.7	0.90
	7.9	15	10	41.4	20.7	0.90
	9.5	10	9	41.4	20.7	0.90
Standard	2.1	40	30	31.0	15.2	0.62
	2.5	35	25	31.0	15.2	0.62
	3.2	35	25	31.0	15.2	0.62
	4.8	35	25	31.0	15.2	0.62
	6.4	25	20	31.0	15.2	0.62
	7.9	20	15	31.0	15.2	0.62
	9.5	15	10	31.0	15.2	0.62
Service-tempered	3.2	35	30	31.0	3.8	0.52
	4.8	30	30	31.0	3.8	0.52
	6.4	30	25	31.0	3.8	0.52
	9.5	20	15	31.0	3.8	0.52

[a]AHA 1995a.

Table 10–12. Physical and mechanical properties of hardboard siding[a]

Property[b]	Requirement	
Water absorption (based on weight)	12% (max avg/panel)	
Thickness swelling	8% (max avg/panel)	
Weatherability of substrate (max residual swell)	20%	
Weatherability of primed substrate	No checking, erosion, flaking, or objectionable fiber raising; adhesion, less than 3.2 mm (0.125 in.) of coating picked up	
Linear expansion 30% to 90% RH (max)	Thickness range (cm)	Maximum linear expansion (%)
	0.220–0.324	0.36
	0.325–0.375	0.38
	0.376–0.450	0.40
	>0.451	0.40
Nail-head pull-through	667 N (150 lb) (min avg/panel)	
Lateral nail resistance	667 N (150 lb) (min avg/panel)	
Modulus of rupture	12.4 MPa (1,800 lb/in^2) for 9.5, 11, and 12.7 mm (3/8, 7/16, and 1/2 in.) thick (min avg/panel)	
	20.7 MPa (3,000 lb/in^2) for 6.4 mm (1/4 in.) thick (min avg/panel)	
Hardness	2002 N (450 lb) (min avg/panel)	
Impact	229 mm (9 in.) (min avg/panel)	
Moisture content[c]	4% to 9% included, and not more than 3% variance between any two boards in any one shipment or order	

[a]From Youngquist and others 1992.
[b]Refer to ANSI/AHA A135.6 I–1990 for test method for determining information on properties.
[c]Since hardboard is a wood-based material, its moisture content varies with environmental humidity conditions. When the environmental humidity conditions in the area of intended use are a critical factor, the purchaser should specify a moisture content range more restrictive than 4% to 9% so that fluctuation in the moisture content of the siding will be kept to a minimum.

properties of hardboard siding. Hardboard siding products come in a great variety of finishes and textures (smooth or embossed) and in different sizes. For application purposes, the AHA siding classifies into three basic types:

Lap siding—boards applied horizontally, with each board overlapping the board below it

Square edge panels—siding intended for vertical application in full sheets

Shiplap edge panel siding—siding intended for vertical application, with the long edges incorporating shiplap joints

The type of panel dictates the application method. The AHA administers a quality conformance program for hardboard for both panel and lap siding. Participation in this program is voluntary and is open to all (not restricted to AHA members). Under this program, hardboard siding products are tested by an independent laboratory in accordance with product standard ANSI/AHA A135.6. Figure 10–13a provides an example of a grade stamp for a siding product meeting this standard.

Insulation Board—Physical and mechanical properties of insulation board are published in the ASTM C208 standard specification for cellulosic fiber insulation board. Physical

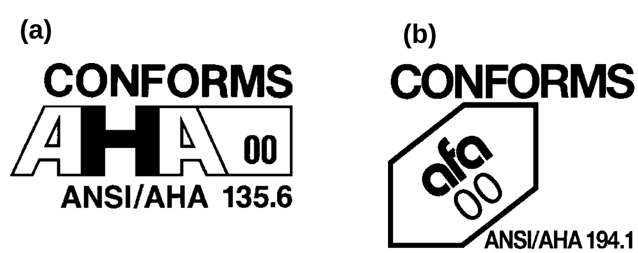


Figure 10–13. Examples of grade stamps: (a) grade stamp for siding conforming to ANSI/AHA A135.6 standard, and (b) grade mark stamp for cellulosic fiberboard products conforming to ANSI/AHA A194.1 standard.

properties are also included in the ANSI standard for cellulosic fiberboard, ANSI/AHA A194.1 (AHA 1985). Insulation board products can be divided into three categories (Suchsland and Woodson 1986): exterior, interior, and industrial.

Exterior products

- Sheathing—board used in exterior construction because of its insulation and noise control qualities, bracing strength, and low price
- Roof decking—three-in-one component that provides roof deck, insulation, and a finished interior ceiling surface; insulation board sheets are laminated together with waterproof adhesive Roof insulation—insulation board designed for use on flat roof decks
- Aluminum siding backer board—fabricated insulation board for improving insulation of aluminum-sided houses

Interior products

- Building board—general purpose product for interior construction
- Ceiling tile—insulation board embossed and decorated for interior use; valued for acoustical qualities; also decorative, nonacoustical tiles
- Sound-deadening board—special product designed to control noise levels in buildings

Industrial products

- Mobile home board
- Expansion joint strips
- Boards for automotive and furniture industries

The AHA administers a quality conformance program for cellulosic fiberboard products including sound-deadening board, roof insulation boards, structural and nonstructural sheathings, backer board, and roof decking in various thicknesses. These products are tested by an independent laboratory in accordance with product standard ANSI/AHA A194.1. An example of the grade mark stamp for these products is shown in Figure 10–13b.

Finishing Techniques

Several techniques are used to finish fiberboard: trimming, sanding, surface treatment, punching, and embossing.

***Trimming*—**Trimming consists of reducing products into standard sizes and shapes. Generally, double-saw trimmers are used to saw the boards. Trimmers consist of overhead-mounted saws or multiple saw drives. Trimmed boards are stacked in piles for future processing.

***Sanding*—**If thickness tolerance is critical, hardboard is sanded prior to finishing. S1S (smooth on one side) boards require this process. Sanding reduces thickness variation and improves surface paintability. Single-head, wide-belt sanders are used with 24- to 36-grit abrasive.

***Surface treatment*—**Surface treatments improve the appearance and performance of boards. Boards are cleaned by spraying with water and then dried at about 240°C (464°F) for 30 seconds. Board surfaces are then modified with paper overlay, paint, or stain or are printed directly on the panel.

***Punching*—**Punching changes boards into the perforated sheets used as peg board. Most punching machines punch three rows of holes simultaneously while the board advances.

***Embossing*—**Embossing consists of pressing the unconsolidated mat of fibers with a textured form. This process results in a slightly contoured board surface that can enhance the resemblance of the board to that of sawn or weathered wood, brick, and other materials.

Specialty Composites

Special-purpose composites are produced to obtain desirable properties like water resistance, mechanical strength, acidity control, and decay and insect resistance. Overlays and veneers can also be added to enhance both structural properties and appearance (Fig. 10–14).

Moisture-Resistant Composites

Sizing agents, wax, and asphalt can be used to make composites resistant to moisture. Sizing agents cover the surface of fibers, reduce surface energy, and render the fibers relatively hydrophobic. Sizing agents can be applied in two ways. In the first method, water is used as a medium to ensure thorough mixing of sizing and fiber. The sizing is forced to precipitate from the water and is fixed to the fiber surface. In the second method, the sizing is applied directly to the fibers. Rosin is a common sizing agent that is obtained from living pine trees, from pine stumps, and as a by-product of kraft pulping of pines. Rosin sizing is added in amounts of less than 3% solids based on dry fiber weight.

Waxes are high molecular weight hydrocarbons derived from crude oil. Wax sizing is used in dry-process fiberboard production; for wet processes, wax is added in solid form or as

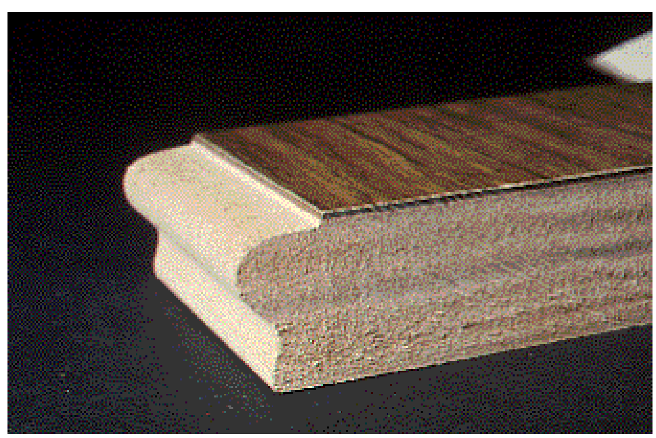


Figure 10–14. Medium-density fiberboard with veneer overlay. Edges can be shaped and finished as required by end product.

an emulsion. Wax sizing tends to lower strength properties to a greater extent than does rosin.

Asphalt is also used to increase water resistance, especially in low-density wet-process insulation board. Asphalt is a black–brown solid or semi-solid material that liquefies when heated. The predominant component of asphalt is bitumen. Asphalt is precipitated onto fiber by the addition of alum.

Flame-Retardant Composites

Two general application methods are available for improving the fire performance of composites with fire-retardant chemicals. One method consists of pressure impregnating the wood with waterborne or organic solventborne chemicals. The second method consists of applying fire-retardant chemical coatings to the wood surface. The impregnation method is usually more effective and longer lasting; however, this technique sometimes causes damage to the wood–adhesive bonds in the composite and results in the degradation of some physical and mechanical properties of the composite. For wood in existing constructions, surface application of fire-retardant paints or other finishes offers a practical method to reduce flame spread.

Preservative-Treated Composites

Wood is highly susceptible to attack by fungi and insects; thus, treatment is essential for maximum durability in adverse conditions.

Composites can be protected from the attack of decay fungi and harmful insects by applying selected chemicals as wood preservatives. The degree of protection obtained depends on the kind of preservative used and the ability to achieve proper penetration and retention of the chemicals. Wood preservative chemicals can be applied using pressure or nonpressure processes. As in the application of fire-retardant chemicals, the application of wood preservatives can sometimes cause damage to wood–adhesive bonds, thus reducing physical and mechanical properties of the composite. Common preservative treatments include chromated copper arsenate (CCA) and boron compounds.

Wood–Nonwood Composites

Interest has burgeoned in combining wood and other raw materials, such as plastics, gypsum, and concrete, into composite products with unique properties and cost benefits (Youngquist and others 1993a, 1993b, 1994; Rowell and others 1997). The primary impetus for developing such products has come from one or more of the following research and development goals:

- Reduce material costs by combining a lower cost material (acting as a filler or extender) with an expensive material
- Develop products that can utilize recycled materials and be recyclable in themselves
- Produce composite products that exhibit specific properties that are superior to those of the component materials alone (for example, increased strength-to-weight ratio, improved abrasion resistance)

Composites made from wood and other materials create enormous opportunities to match product performance to end-use requirements (Youngquist 1995).

Figure 10–15. Laboratory-produced low-density, cement-bonded composite panel. Full-scale panels such as these are used in construction.

Inorganic–Bonded Composites

Inorganic-bonded wood composites have a long and varied history that started with commercial production in Austria in 1914. A plethora of building materials can be made using inorganic binders and lignocellulosics, and they run the normal gamut of panel products, siding, roofing tiles, and precast building members (Fig. 10–15).

Inorganic-bonded wood composites are molded products or boards that contain between 10% and 70% by weight wood particles or fibers and conversely 90% to 30% inorganic binder. Acceptable properties of an inorganic-bonded wood composite can be obtained only when the wood particles are fully encased with the binder to make a coherent material. This differs considerably from the technique used to manufacture thermosetting-resin-bonded boards where flakes or particles are “spot welded” by a binder applied as a finely distributed spray or powder. Because of this difference and because hardened inorganic binders have a higher density than that of most thermosetting resins, the required amount of inorganic binder per unit volume of composite material is much higher than that of resin-bonded wood composites. The properties of inorganic-bonded wood composites are significantly influenced by the amount and nature of the inorganic binder and the woody material as well as the density of the composites.

Inorganic binders fall into three main categories: gypsum, magnesia cement, and Portland cement. Gypsum and magnesia cement are sensitive to moisture, and their use is generally restricted to interior applications. Composites bonded with Portland cement are more durable than those bonded with gypsum or magnesia cement and are used in both interior and exterior applications. Inorganic-bonded composites are made by blending proportionate amounts of lignocellulosic fiber with inorganic materials in the presence of water and allowing the inorganic material to cure or "set up" to make a rigid composite. All inorganic-bonded composites are very resistant to deterioration, particularly by insects, vermin, and fire.

A unique feature of inorganic-bonded composites is that their manufacture is adaptable to either end of the cost and technology spectrum. This is facilitated by the fact that no heat is required to cure the inorganic material. For example, in the Philippines, Portland cement-bonded composites are mostly fabricated using manual labor and are used in low-cost housing. In Japan, the fabrication of these composites is automated, and they are used in very expensive modular housing.

The versatility of manufacture makes inorganic-bonded composites ideally suited to a variety of lignocellulosic materials. With a very small capital investment and the most rudimentary of tools, satisfactory inorganic-bonded lignocellulosic composite building materials can be produced on a small scale using mostly unskilled labor. If the market for such composites increases, technology can be introduced to increase manufacturing throughput. The labor force can be trained concurrently with the gradual introduction of more sophisticated technology.

Gypsum-Bonded Composites

Gypsum can be derived by mining from natural sources or obtained as a byproduct of flue gas neutralization. Flue gas gypsum, now being produced in very large quantities in the United States because of Clean Air Act regulations, is the result of introducing lime into the combustion process to reduce sulfur dioxide emissions. In 1995, more than 100 power plants throughout the United States were producing gypsum. Flue gas gypsum can be used in lieu of mined gypsum.

Gypsum panels are frequently used to finish interior wall and ceiling surfaces. In the United States, these products are generically called "dry wall" because they replace wet plaster systems. To increase the bending strength and stiffness, gypsum panels are frequently wrapped in paper, which provides a tension surface. An alternative to wrapping gypsum with fiber is to place the fiber within the panel, as several U.S. and European firms are doing with recycled paper fiber. There is no technical reason that other lignocellulosics cannot be used in this way. Gypsum is widely available and does not have the highly alkaline environment of cement.

Gypsum panels are normally made from a slurry of gypsum, water, and lignocellulosic fiber. In large-scale production, the slurry is extruded onto a belt, which carries the slurry through a drying oven to evaporate water and facilitate cure of the gypsum. The panel is then cut to length and trimmed if necessary.

Magnesia-Cement-Bonded Composites

Fewer boards bonded with magnesia cement have been produced than cement- or gypsum-bonded panels, mainly because of price. However, magnesia cement does offer some manufacturing advantages over Portland cement. First, the various sugars in lignocellulosics apparently do not have as much effect on the curing and bonding of the binder. Second, magnesia cement is reported to be more tolerant of high water content during production. This opens up possibilities to use lignocellulosics not amenable to Portland cement composites, without leaching or other modification, and to use alternative manufacturing processes and products. Although composites bonded with magnesia cement are considered water-sensitive, they are much less so than gypsum-bonded composites.

One successful application of magnesia cement is a low-density panel made for interior ceiling and wall applications. In the production of this panel product, wood wool (excelsior) is laid out in a low-density mat. The mat is then sprayed with an aqueous solution of magnesia cement, pressed, and cut into panels.

In Finland, magnesia-cement-bonded particleboard is manufactured using a converted conventional particleboard plant. Magnesia oxide is applied to the lignocellulosic particles in a batch blender along with other chemicals and water. Depending on application and other factors, boards may be cold- or hot-pressed.

Other processes have been suggested for manufacturing magnesia-cement-bonded composites. One application may be to spray a slurry of magnesia cement, water, and lignocellulosic fiber onto existing structures as fireproofing. Extrusion into a pipe-type profile or other profiles is also possible.

Portland-Cement-Bonded Composites

The most apparent and widely used inorganic-bonded composites are those bonded with Portland cement. Portland cement, when combined with water, immediately reacts in a process called hydration to eventually solidify into a solid stone-like mass. Successfully marketed Portland-cement-bonded composites consist of both low-density products made with excelsior and high-density products made with particles and fibers. General mechanical property values for a low density cement–wood excelsior product are given in Table 10–13.

The low-density products may be used as interior ceiling and wall panels in commercial buildings. In addition to the advantages described for low-density magnesia-bonded composites, low-density composites bonded with Portland cement offer sound control and can be quite decorative. In some parts of the world, these panels function as complete wall and roof decking systems. The exterior of the panels is stuccoed, and the interior is plastered. High-density panels can be used as flooring, roof sheathing, fire doors, load-bearing walls, and cement forms. Fairly complex molded shapes can be molded or extruded, such as decorative roofing tiles or non-pressure pipes.

Table 10–13. General properties of low-density cement–wood composites fabricated using an excelsior-type particle[a,b]

Property	From	To
Bending strength	1.7 MPa (250 lb/in^2)	5.5 MPa (800 lb/in^2)
Modulus of elasticity	621 MPa (0.9 × 10^5 lb/in^2)	1,241 MPa (1.8 × 10^5 lb/in^2)
Tensile strength	0.69 MPa (100 lb/in^2)	4.1 MPa (600 lb/in^2)
Compression strength	0.69 MPa (100 lb/in^2)	5.5 MPa (800 lb/in^2)
Shear[c]	0.69 MPa (100 lb/in^2)	1.4 MPa (200 lb/in^2)
E/*G*[d]	40	100

[a]Data represent compilation of raw data from variety of sources for range of board properties. Variables include cement–wood mix, particle configuration, board density, and the forming and curing methods.
[b]Specific gravity range, 0.5 to 1.0.
[c]Shear strength data are limited to a small sample of excelsior boards having a specific gravity of 0.5 to 0.65.
[d]*E*/*G* is ratio of bending modulus of elasticity to modulus of rigidity or shear modulus. For wood, this ratio is often assumed to be around 16.

Problems and Solutions

Although the entire sphere of inorganic-bonded lignocellulosic composites is attractive, and cement-bonded composites are especially so, the use of cement involves limitations and tradeoffs. Marked embrittlement of the lignocellulosic component is known to occur and is caused by the alkaline environment provided by the cement matrix. In addition, hemicellulose, starch, sugar, tannins, and lignin, all to a varying degree, affect the cure rate and ultimate strength of these composites. To make strong and durable composites, measures must be taken to ensure long-term stability of the lignocellulosic in the cement matrix. To overcome these problems, various schemes have been developed. The most common is leaching, whereby the lignocellulosic is soaked in water for 1 or 2 days to extract some of the detrimental components. However, in some parts of the world, the water containing the leachate is difficult to dispose of. Low water–cement ratios are helpful, as is the use of curing accelerators like calcium carbonate. Conversely, low alkali cements have been developed, but they are not readily available throughout the world. Two other strategies are natural pozzolans and carbon dioxide treatment.

Natural Pozzolans—Pozzolans are defined as siliceous or siliceous and aluminous materials that can react chemically with calcium hydroxide (lime) at normal temperatures in the presence of water to form cement compounds (ASTM 1988). Some common pozzolanic materials include volcanic ash, fly ash, rice husk ash, and condensed silica fume. All these materials can react with lime at normal temperatures to make a natural water-resistant cement.

In general, when pozzolans are blended with Portland cement, they increase the strength of the cement but slow the cure time. More important, pozzolans decrease the alkalinity of Portland cement, which indicates that adding lignocellulosic-based material (rice husk ash) to cement-bonded lignocellulosic composites may be advantageous.

Carbon Dioxide Treatment—In the manufacture of a cement-bonded lignocellulosic composite, the cement hydration process normally requires from 8 to 24 h to develop sufficient board strength and cohesiveness to permit the release of consolidation pressure. By exposing the cement to carbon dioxide, the initial hardening stage can be reduced to less than 5 min. This phenomenon results from the chemical reaction of carbon dioxide with calcium hydroxide to form calcium carbonate and water.

Reduction of initial cure time of the cement-bonded lignocellulosic composite is not the only advantage of using carbon dioxide injection. Certain species of wood have varying amounts of sugars and tannins that interfere with the hydration or setting of Portland cement. Research has shown that the use of carbon dioxide injection reduces the likelihood of these compounds to inhibit the hydration process, thus allowing the use of a wider range of species in these composites. In addition, research has demonstrated that composites treated with carbon dioxide can be twice as stiff and strong as untreated composites (Geimer and others 1992). Finally, carbon-dioxide-treated composites do not experience efflorescence (migration of calcium hydroxide to surface of material), so the appearance of the surface of the final product is not changed over time.

Wood Fiber–Thermoplastic Composites

As described elsewhere in this chapter, the use of lignocellulosic materials with thermosetting polymeric materials, like phenol- or urea-formaldehyde, in the production of composites has a long history. The use of lignocellulosics with thermoplastics, however, is a more recent innovation. Broadly defined, a thermoplastic softens when heated and hardens when cooled. Thermoplastics selected for use with lignocellulosics must melt or soften at or below the degradation point of the lignocellulosic component, normally 200°C to 220°C (392°F to 428°F). These thermoplastics include polypropylene, polystyrene, vinyls, and low- and high-density polyethylenes.

Wood flour is a readily available resource that can be used as a filler in thermoplastic composites. Wood flour is processed commercially, often from post-industrial materials such as planer shavings, chips, and sawdust. Several grades are available depending upon wood species and particle size. Wood fibers, although more difficult to process compared with wood flour, can lead to superior composite properties and act more as a reinforcement than as a filler. A wide variety of wood fibers are available from both virgin and recycled resources.

Other materials can be added to affect processing and product performance of wood–thermoplastic composites. These additives can improve bonding between the thermoplastic and wood component (for example, coupling agents), product performance (impact modifiers, UV stabilizers, flame retardants), and processability (lubricants).

Several considerations must be kept in mind when processing wood with thermoplastics. Moisture can disrupt many thermoplastic processes, resulting in poor surface quality, voids, and unacceptable parts. Materials must either be predried or vented equipment must be used to remove moisture. The low degradation temperature of wood must also be considered. As a general rule, melt temperatures should be kept below 200°C (392°F), except for short periods. Higher temperatures can result in the release of volatiles, discoloration, odor, and embrittlement of the wood component.

There are two main strategies for processing thermoplastics in lignocellulosic composites (Youngquist and others 1993b). In the first, the lignocellulosic component serves as a reinforcing agent or filler in a continuous thermoplastic matrix. In the second, the thermoplastic serves as a binder to the majority lignocellulosic component. The presence or absence of a continuous thermoplastic matrix may also determine the processability of the composite material. In general, if the matrix is continuous, conventional thermoplastic processing equipment may be used to process composites; however, if the matrix is not continuous, other processes may be required. For the purpose of discussion, we present these two scenarios for composites with high and low thermoplastic content.

Composites With High Thermoplastic Content

In composites with high thermoplastic content, the thermoplastic component is in a continuous matrix and the lignocellulosic component serves as a reinforcement or filler (Fig. 10–16). In the great majority of reinforced thermoplastic composites available commercially, inorganic materials (for example, glass, clays, and minerals) are used as reinforcements or fillers. Lignocellulosic materials offer some advantages over inorganic materials; they are lighter, much less abrasive, and renewable. As a reinforcement, lignocellulosics can stiffen and strengthen the thermoplastic and can improve thermal stability of the product compared with that of unfilled material.

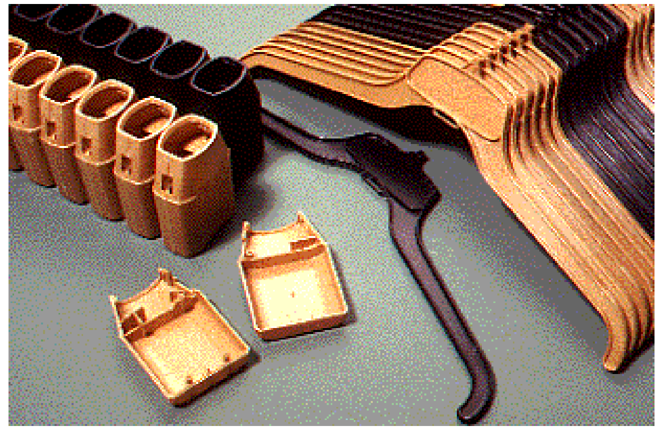


Figure 10–16. The use of lignocelulosics as reinforcing fillers allows thermoplastics to be molded into a wide variety of shapes and forms.

Thermoplastics in pellet form have bulk density in the range of 500 to 600 kg/m^3 (31 to 37 lb/ft^3). Lignocellulosics typically have an uncompacted bulk density of 25 to 250 kg/m^3 (1.6 to 16 lb/ft^3). Wood fibers are at the low end of the lignocellulosic bulk density continuum and wood flours at the high end. Although processing of wood flour in thermoplastics is relatively easy, the low bulk density and difficulty of dispersing fibrous materials make thermoplastics more difficult to compound. More intensive mixing and the use of special feeding equipment may be necessary to handle longer fibers.

The manufacture of thermoplastic composites is usually a two-step process. The raw materials are first mixed together, and the composite blend is then formed into a product. The combination of these steps is called in-line processing, and the result is a single processing step that converts raw materials to end products. In-line processing can be very difficult because of control demands and processing trade-offs. As a result, it is often easier and more economical to separate the processing steps.

Compounding is the feeding and dispersing of the lignocellulosic component in a molten thermoplastic to produce a homogeneous material. Various additives are added and moisture is removed during compounding. Compounding may be accomplished using either batch mixers (for example, internal and thermokinetic mixers) or continuous mixers (for example, extruders and kneaders). Batch systems allow closer control of residence time, shear, and temperature than do continuous systems. Batch systems are also more appropriate for operations consisting of short runs and frequent change of materials. On the other hand, continuous systems are less operator-dependent than are batch systems and have less batch-to-batch differences (Anon. 1997).

The compounded material can be immediately pressed or shaped into an end product while still in its molten state or pelletized into small, regular pellets for future reheating and forming. The most common types of product-forming methods for wood–thermoplastic composites involve forcing molten material through a die (sheet or profile extrusion) into a cold mold (injection molding) or pressing in calenders (calendering) or between mold halves (thermoforming and compression molding).

Properties of wood–plastic composites can vary greatly depending upon such variables as type, form, and weight fractions of constituents, types of additives, and processing history. Table 10–14 shows some of the properties for several unfilled polypropylene and wood–polypropylene composites.

Composites with high thermoplastic content are not without tradeoffs. Impact resistance of such composites decreases compared with that of unfilled thermoplastics, and these composites are also more sensitive to moisture than unfilled material or composites filled with inorganic material. From a practical standpoint, however, the thermoplastic component usually makes the temperature sensitivity of the composite more significant than any change in properties brought about by moisture absorption.

Composites With Low Thermoplastic Content

Composites with low thermoplastic content can be made in a variety of ways. In the simplest form, the thermoplastic component acts much the same way as a thermosetting resin; that is, as a binder to the lignocellulosic component. An alternative is to use the thermoplastic in the form of a textile fiber. The thermoplastic textile fiber enables a variety of lignocellulosics to be incorporated into a low-density, nonwoven, textile-like mat. The mat may be a product in itself, or it may be consolidated into a high-density product.

Experimentally, low-thermoplastic-content composites have been made that are very similar to conventional lignocellulosic composites in many performance characteristics (Youngquist and others 1993b). In their simplest form, lignocellulosic particles or fibers can be dry-blended with thermoplastic granules, flakes, or fibers and pressed into panel products.

Because the thermoplastic component remains molten when hot, different pressing strategies must be used than when thermosetting binders are used. Two options have been developed to accommodate these types of composites. In the first, the material is placed in the hot press at ambient temperature. The press then closes and consolidates the material, and heat is transferred through conduction to melt the thermoplastic component, which flows around the lignocellulosic component. The press is then cooled, “freezing” the thermoplastic so that the composite can be removed from the press. Alternatively, the material can be first heated in an oven or hot press. The hot material is then transferred to a cool press where it is quickly consolidated and cooled to make a rigid panel. Some commercial nonstructural lignocellulosic–thermoplastic composites are made in this way.

Nonwoven Textile-Type Composites

In contrast to high-thermoplastic-content and conventional low-thermoplastic-content composites, nonwoven textile-type composites typically require long fibrous materials for their manufacture. These fibers might be treated jute or kenaf, but more typically they are synthetic thermoplastic materials. Nonwoven processes allow and tolerate a wider range of lignocellulosic materials and synthetic fibers, depending on product applications. After fibers are dry-blended, they are air-laid into a continuous, loosely consolidated mat. The mat is then passed through a secondary operation in which the fibers are mechanically entangled or otherwise bonded together. This low-density mat may be a product in itself, or the mat may be shaped and densified in a thermoforming step (Youngquist and others 1993b).

If left as low density and used without significant modification by post-processing, the mats have a bulk density of

Table 10–14. Mechanical properties of wood–polypropylene composites[a,b]

		Tensile			Flexural		Izod impact energy		
Composite[c]	Density (g/cm^3 (lb/ft^3))	Strength (MPa (lb/in^2))	Modulus (GPa (lb/in^2))	Elongation (%)	Strength (MPa (lb/in^2))	Modulus (GPa (lb/in^2))	Notched (J/m (ft-lbf/in))	Unnotched (J/m (ft-lbf/in))	Heat deflection temperature (°C (°F))
Polypropylene	0.9 (56.2)	28.5 (4,130)	1.53 (221,000)	5.9	38.3 (5,550)	1.19 (173,000)	20.9 (0.39)	656 (12.3)	57 (135)
PP + 40% wood flour	1.05 (65.5)	25.4 (3,680)	3.87 (561,000)	1.9	44.2 (6,410)	3.03 (439,000)	22.2 (0.42)	73 (1.4)	89 (192)
PP + 40% hardwood fiber	1.03 (64.3)	28.2 (4,090)	4.20 (609,000)	2.0	47.9 (6,950)	3.25 (471,000)	26.2 (0.49)	91 (1.7)	100 (212)
PP + 40% hardwood fiber + 3% coupling agent	1.03 (64.3)	52.3 (7,580)	4.23 (613,000)	3.2	72.4 (10,500)	3.22 (467,000)	21.6 (0.41)	162 (3.0)	105 (221)

[a]Unpublished data.
[b]Properties measured according to ASTM standards for plastics.
[c]PP is polypropylene; percentages based on weight.

50 to 250 kg/m^3 (3 to 16 lb/ft^3). These products are particularly well known in the consumer products industry, where nonwoven technology is used to make a variety of absorbent personal care products, wipes, and other disposable items. The products are made from high-quality pulps in conjunction with additives to increase absorptive properties. A much wider variety of lignocellulosics can be used for other applications, as described in the following text.

One interesting application for low-density nonwoven mats is for mulch around newly planted seedlings. The mats provide the benefits of natural mulch; in addition, controlled-release fertilizers, repellents, insecticides, and herbicides can be added to the mats. The addition of such chemicals could be based on silvicultural prescriptions to ensure seedling survival and early development on planting sites where severe nutritional deficiencies, animal damage, insect attack, and weeds are anticipated.

Low-density nonwoven mats can also be used to replace dirt or sod for grass seeding around new home sites or along highway embankments. Grass seed can be incorporated directly into the mat. These mats promote seed germination and good moisture retention. Low-density mats can also be used for filters. The density can be varied, depending on the material being filtered and the volume of material that passes through the mat per unit of time.

High-density fiber mats can be defined as composites that are made using the nonwoven mat process and then formed into rigid shapes by heat and pressure. To ensure good bonding, the lignocellulosic can be precoated with a thermosetting resin such as phenol–formaldehyde, or it can be blended with synthetic fibers, thermoplastic granules, or any combination of these materials. High-density fiber mats can typically be pressed into products having a specific gravity of 0.60 to 1.40. Table 10–15 presents mechanical and physical property

Table 10–15. Properties of nonwoven web composite panels with specific gravity of 1.0[a]

	Formulation[b]		
Property	90H/10PE	90H/10PP	80H/10PE/PR
Static bending MOR, MPa (lb/in^2)	23.3 (3,380)	25.5 (3,700)	49.3 (7,150)
Cantilever bending MOR, MPa (lb/in^2)	21.1 (3,060)	27.1 (3,930)	45.6 (6,610)
Static bending MOE, GPa ($\times 10^3$ lb/in^2)	2.82 (409)	2.99 (434)	3.57 (518)
Dynamic MOE, GPa ($\times 10^3$ lb/in^2)	4.75 (689)	5.27 (764)	5.52 (800)
Tensile strength, MPa (lb/in^2)	13.5 (1,960)	12.5 (1,810)	27.7 (4,020)
Tensile MOE, GPa ($\times 10^3$ lb/in^2)	3.87 (561)	420 (609)	5.07 (735)
Internal bond, MPa (lb/in^2)	0.14 (20)	0.28 (41)	0.81 (120)
Impact energy, J (ft·lbf)	26.7 (19.7)	21.5 (15.9)	34.3 (25.3)
Water-soak, 24 h			
Thickness swell, %	60.8	40.3	21.8
Water absorption, %	85.0	54.7	45.1
Water boil, 2 h			
Thickness swell, %	260.1	77.5	28.2
Water absorption, %	301.6	99.5	55.7
Linear expansion[c]			
Ovendry to			
30% RH, %	0.13	0.00	0.55
65% RH, %	0.38	0.25	0.76
90% RH, %	0.81	0.78	0.93
Equilibrium MC at			
30% RH, %	3.4	3.4	3.4
65% RH, %	6.4	6.2	6.3
90% RH, %	15.6	14.9	14.1

[a]From Youngquist and others 1992.
[b]Values connected by solid line are not statistically different at 0.05 significance level. 90H/10PE, 90% hemlock and 10% polyester; 90H/10PP, 90% hemlock and 10% polypropylene; 80H/10PE/10PR, 80% hemlock, 10% polyester, and 10% phenolic resin.
[c]RH = relative humidity.

data for nonwoven web composite panels with a specific gravity of 1.0 for three different formulations of wood, synthetic fibers, and phenolic resin. After thermoforming, the products possess good temperature resistance. Because longer fibers are used, these products exhibit better mechanical properties than those obtained with high-thermoplastic-content composites; however, the high lignocellulosic content leads to increased moisture sensitivity.

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Chapter 11

Glued Structural Members

Russell C. Moody, Roland Hernandez, and Jen Y. Liu

Contents

Glued structural members are manufactured in a variety of configurations. Structural composite lumber (SCL) products consist of small pieces of wood glued together into sizes common for solid-sawn lumber. Glued-laminated timber (glulam) is an engineered stress-rated product that consists of two or more layers of lumber in which the grain of all layers is oriented parallel to the length of the lumber. Glued structural members also include lumber that is glued to panel products, such as box beams and I-beams, and structural sandwich construction.

Structural Composite Lumber

Structural composite lumber was developed in response to the increasing demand for high quality lumber at a time when it was becoming difficult to obtain this type of lumber from the forest resource. Structural composite lumber products are characterized by smaller pieces of wood glued together into sizes common for solid-sawn lumber.

One type of SCL product is manufactured by laminating veneer with all plies parallel to the length. This product is called laminated veneer lumber (LVL) and consists of specially graded veneer. Another type of SCL product consists of strands of wood or strips of veneer glued together under high pressures and temperatures. Depending upon the component material, this product is called laminated strand lumber (LSL), parallel strand lumber (PSL), or oriented strand lumber (OSL) (Fig. 11–1). These types of SCL products can be manufactured from raw materials, such as aspen or other underutilized species, that are not commonly used for structural applications. Different widths of lumber can be ripped from SCL for various uses.

Structural composite lumber is a growing segment of the engineered wood products industry. It is used as a replacement for lumber in various applications and in the manufacture of other engineered wood products, such as prefabricated wood I-joists, which take advantage of engineering design values that can be greater than those commonly assigned to sawn lumber.

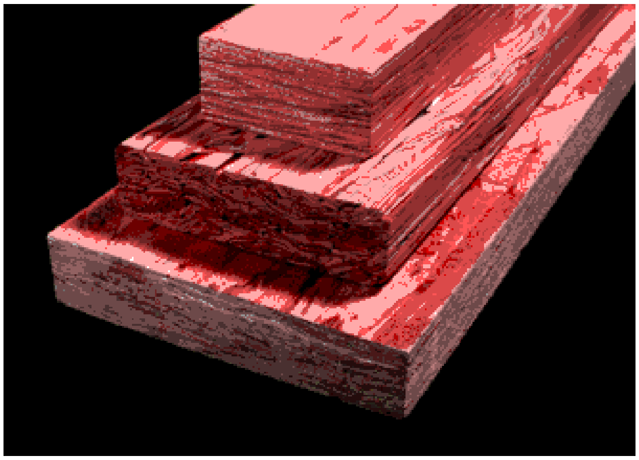

Figure 11–1. Examples of three types of SCL (top to bottom): laminated veneer lumber (LVL), parallel strand lumber (PSL), and oriented strand lumber (OSL).

Types

Laminated Veneer Lumber

Work in the 1940s on LVL targeted the production of high strength parts for aircraft structures using Sitka spruce veneer. Research on LVL in the 1970s was aimed at defining the effects of processing variables for veneer up to 12.7 mm (1/2 in.) thick. In the 1990s, production of LVL uses veneers 3.2 to 2.5 mm (1/8 to 1/10 in.) thick, which are hot pressed with phenol-formaldehyde adhesive into lengths from 2.4 to 18.3 m (8 to 60 ft) or more.

The veneer for the manufacture of LVL must be carefully selected for the product to achieve the desired engineering properties. The visual grading criteria of PS 1–95 (NIST 1995) are sometimes used but are generally not adequate without additional grading. Veneers are often sorted using ultrasonic testing to ensure that the finished product will have the desired engineering properties.

End joints between individual veneers may be staggered along the product to minimize their effect on strength. These end joints may be butt joints, or the veneer ends may overlap for some distance to provide load transfer. Some producers provide structural end joints in the veneers using either scarf or fingerjoints. Laminated veneer lumber may also be made in 2.4-m (8-ft) lengths, having no end joints in the veneer; longer pieces are then formed by end jointing these pieces to create the desired length.

Sheets of LVL are commonly produced in 0.6- to 1.2-m (2- to 4-ft) widths in a thickness of 38 mm (1.5 in.). Continuous presses can be used to form a potentially endless sheet, which is cut to the desired length. Various widths of lumber can be manufactured at the plant or the retail facility.

Parallel Strand Lumber

Parallel strand lumber (PSL) is defined as a composite of wood strand elements with wood fibers primarily oriented along the length of the member. The least dimension of the strands must not exceed 6.4 mm (0.25 in.), and the average length of the strands must be a minimum of 150 times the least dimension. In 1997, one commercial product in the United States was classified as PSL.

Parallel strand lumber is manufactured using veneer about 3 mm (1/8 in.) thick, which is then clipped into strands about 19 mm (3/4 in.) wide. These strands are commonly at least 0.6 m (24 in.) long. The manufacturing process was designed to use the material from roundup of the log in the veneer cutting operation as well as other less than full-width veneer. Thus, the process can utilize waste material from a plywood or LVL operation. Species commonly used for PSL include Douglas-fir, southern pines, western hemlock, and yellow-poplar, but there are no restrictions on using other species.

The strands are coated with a waterproof structural adhesive, commonly phenol-resorcinol formaldehyde, and oriented in a press using special equipment to ensure proper orientation and distribution. The pressing operation results in densification of the material, and the adhesive is cured using microwave technology. Billets larger than those of LVL are commonly produced; a typical size is 0.28 by 0.48 m (11 by 19 in.). This product can then be sawn into smaller pieces, if desired. As with LVL, a continuous press is used so that the length of the product is limited by handling restrictions.

Laminated Strand Lumber and Oriented Strand Lumber

Laminated strand lumber (LSL) and oriented strand lumber (OSL) products are an extension of the technology used to produce oriented strandboard (OSB) structural panels. One type of LSL uses strands that are about 0.3 m (12 in.) long, which is somewhat longer than the strands commonly used for OSB. Waterproof adhesives are used in the manufacture of LSL. One type of product uses an isocyanate type of adhesive that is sprayed on the strands and cured by steam injection. This product needs a greater degree of alignment of the strands than does OSB and higher pressures, which result in increased densification.

Advantages and Uses

In contrast with sawn lumber, the strength-reducing characteristics of SCL are dispersed within the veneer or strands and have much less of an effect on strength properties. Thus, relatively high design values can be assigned to strength properties for both LVL and PSL. Whereas both LSL and OSL have somewhat lower design values, they have the advantage of being produced from a raw material that need not be in a log size large enough for peeling into veneer. All SCL products are made with structural adhesives and are dependent upon a minimum level of strength in these bonds. All SCL products are made from veneers or strands that are dried to a moisture content that is slightly less than that for most service conditions. Thus, little change in moisture content will occur in many protected service conditions.

When used indoors, this results in a product that is less likely to warp or shrink in service. However, the porous nature of both LVL and PSL means that these products can quickly absorb water unless they are provided with some protection.

All types of SCL products can be substituted for sawn lumber products in many applications. Laminated veneer lumber is used extensively for scaffold planks and in the flanges of prefabricated I-joists, which takes advantage of the relatively high design properties. Both LVL and PSL beams are used as headers and major load-carrying elements in construction. The LSL and OSL products are used for band joists in floor construction and as substitutes for studs and rafters in wall and roof construction. Various types of SCL are also used in a number of nonstructural applications, such as the manufacture of windows and doors.

Standards and Specifications

The ASTM D5456 (ASTM 1997a) standard provides methods to develop design properties for SCL products as well as requirements for quality assurance during production. Each manufacturer of SCL products is responsible for developing the required information on properties and ensuring that the minimum levels of quality are maintained during production. An independent inspection agency is required to monitor the quality assurance program.

Unlike lumber, no standard grades or design stresses have been established for SCL. Each manufacturer may have unique design properties and procedures. Thus, the designer should consult information provided by the manufacturer.

Glulam

Structural glued-laminated timber (glulam) is one of the oldest glued engineered wood products. Glulam is an engineered, stress-rated product that consists of two or more layers of lumber that are glued together with the grain of all layers, which are referred to as laminations, parallel to the length. Glulam is defined as a material that is made from suitably selected and prepared pieces of wood either in a straight or curved form, with the grain of all pieces essentially parallel to the longitudinal axis of the member. The maximum lamination thickness permitted is 50 mm (2 in.), and the laminations are typically made of standard 25- or 50-mm- (nominal 1- or 2-in.-) thick lumber. North American standards require that glulam be manufactured in an approved manufacturing plant. Because the lumber is joined end to end, edge to edge, and face to face, the size of glulam is limited only by the capabilities of the manufacturing plant and the transportation system.

Douglas Fir–Larch, Southern Pine, Hem–Fir, and Spruce–Pine–Fir (SPF) are commonly used for glulam in the United States. Nearly any species can be used for glulam timber, provided its mechanical and physical properties are suitable and it can be properly glued. Industry standards cover many softwoods and hardwoods, and procedures are in place for including other species.

Advantages

Compared with sawn timbers as well as other structural materials, glulam has several distinct advantages in size capability, architectural effects, seasoning, variation of cross sections, grades, and effect on the environment.

Size Capabilities—Glulam offers the advantage of the manufacture of structural timbers that are much larger than the trees from which the component lumber was sawn. In the past, the United States had access to large trees that could produce relatively large sawn timbers. However, the present trend is to harvest smaller diameter trees on much shorter rotations, and nearly all new sawmills are built to accommodate relatively small logs. By combining the lumber in glulam, the production of large structural elements is possible. Straight members up to 30 m (100 ft) long are not uncommon and some span up to 43 m (140 ft). Sections deeper than 2 m (7 ft) have been used. Thus, glulam offers the potential to produce large timbers from small trees.

Architectural Effects—By curving the lumber during the manufacturing process, a variety of architectural effects can be obtained that are impossible or very difficult with other materials. The degree of curvature is controlled by the thickness of the laminations. Thus, glulam with moderate curvature is generally manufactured with standard 19-mm- (nominal 1-in.-) thick lumber. Low curvatures are possible with standard 38-mm (nominal 2-in.) lumber, whereas 13 mm (1/2 in.) or thinner material may be required for very sharp curves. As noted later in this chapter, the radius of curvature is limited to between 100 and 125 times the lamination thickness.

Seasoning Advantages—The lumber used in the manufacture of glulam must be seasoned or dried prior to use, so the effects of checking and other drying defects are minimized. In addition, design can be on the basis of seasoned wood, which permits greater design values than can be assigned to unseasoned timber.

Varying Cross Sections—Structural elements can be designed with varying cross sections along their length as determined by strength and stiffness requirements. The beams in Figure 11–2 show how the central section of the beam can be made deeper to account for increased structural requirements in this region of the beam. Similarly, arches often have varying cross sections as determined by design requirements.

Varying Grades—One major advantage of glulam is that a large quantity of lower grade lumber can be used within the less highly stressed laminations of the beams. Grades are often varied within the beams so that the highest grades are used in the highly stressed laminations near the top and bottom and the lower grade for the inner half or more of the beams. Species can also be varied to match the structural requirements of the laminations.

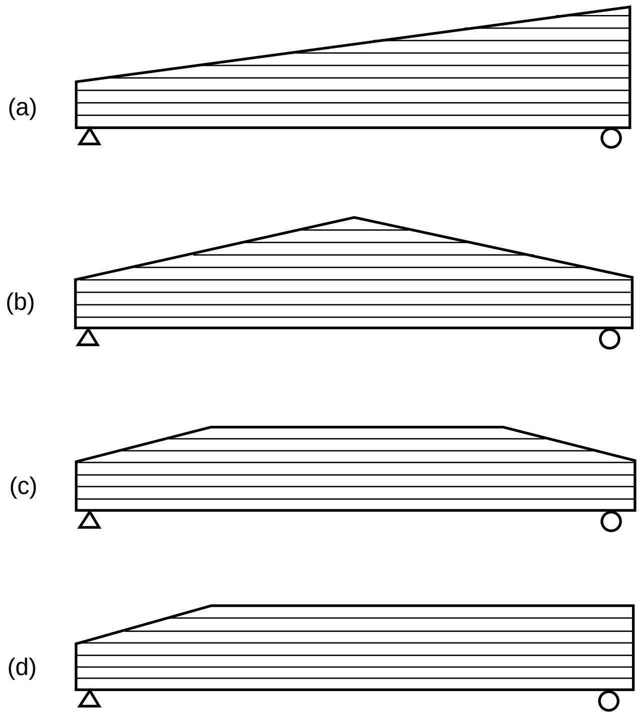

Figure 11–2. Glulam timbers may be (a) single tapered, (b) double tapered, (c) tapered at both ends, or (d) tapered at one end.

Environmentally Friendly—Much is being written and discussed regarding the relative environmental effects of various materials. Several analyses have shown that the renewability of wood, its relatively low requirement for energy during manufacture, its carbon storage capabilities, and its recyclability offer potential long-term environmental advantages over other materials. Although aesthetics and economic considerations usually are the major factors influencing material selection, these environmental advantages may increasingly influence material selection.

The advantages of glulam are tempered by certain factors that are not encountered in the production of sawn timber. In instances where solid timbers are available in the required size, the extra processing in making glulam timber usually increases its cost above that of sawn timbers. The manufacture of glulam requires special equipment, adhesives, plant facilities, and manufacturing skills, which are not needed to produce sawn timbers. All steps in the manufacturing process require care to ensure the high quality of the finished product. One factor that must be considered early in the design of large straight or curved timbers is handling and shipping.

History

Glulam was first used in Europe in the construction of an auditorium in Basel, Switzerland, in 1893, which is often cited as the first known significant use of this product. It was patented as the "Hertzer System" and used adhesives that, by today's standards, are not waterproof. Thus, applications were limited to dry-use conditions. Improvements in adhesives during and following World War I stimulated additional interest in Europe in regard to using glulam in aircraft and building frames.

In the United States, one of the first examples of glulam arches designed and built using engineering principles is in a building erected in 1934 at the USDA Forest Service, Forest Products Laboratory, Madison, Wisconsin (Fig. 11–3). The founder of a company that produced many of these initial buildings in the United States was a German immigrant who transferred the technology to his manufacturing facility in Peshtigo, Wisconsin. Applications included gymnasiums, churches, halls, factories, and barns. Several other companies based on the same technology were soon established.

World War II stimulated additional interest and the development of synthetic resin adhesives that were waterproof. This permitted the use of glulam timber in bridges and other exterior applications that required preservative treatment. By the early 1950s, there were at least a dozen manufacturers of glulam timber in the United States, who joined together to form the American Institute of Timber Construction (AITC). In 1963, this association produced the first national manufacturing standard. The AITC continues to prepare, update, and distribute industry standards for manufacture and design of glulam. By the mid-1990s, about 30 manufacturing plants across the United States and Canada were qualified to produce glulam, according to the requirements of the AITC standard.

From the mid-1930s through the 1980s, nearly all glulam production was used domestically. During the 1990s, the export market was developed and significant quantities of material were shipped to Pacific Rim countries, mainly Japan.

Types of Glulam Combinations

Bending Members

The configuring of various grades of lumber to form a glulam cross section is commonly referred to as a glulam combination. Glulam combinations subjected to flexural loads, called bending combinations, were developed to provide the most efficient and economical section for resisting bending stress caused by loads applied perpendicular to the wide faces of the laminations. This type of glulam is commonly referred to as a horizontally laminated member. Lower grades of laminating lumber are commonly used for the center portion of the combination, or core, where bending stress is low, while a higher grade of material is placed on the outside faces where bending stress is relatively high. To optimize the bending stiffness of this type of glulam member, equal amounts of high quality laminations on the outside faces should be included to produce a "balanced" combination. To optimize bending strength, the combination can be "unbalanced" with more high quality laminations placed on the tension side of the member compared with the quality used on the compression side. For high quality lumber placed on the tension side of the glulam combination, stringent requirements are

Figure 11–3. Erected in 1934 at the Forest Products Laboratory in Madison, Wisconsin, this building is one of the first constructed with glued-laminated timbers arched, designed, and built using engineering principles.

placed on knot size, slope of grain, and lumber stiffness. For compression-side laminations, however, knot size and slope-of-grain requirements are less stringent and only lumber stiffness is given high priority. In the case where the glulam member is used over continuous supports, the combination would need to be designed as a balanced member for strength and stiffness because of the exposure of both the top and bottom of the beam to tensile stresses. The knot and slope-of-grain requirements for this type of combination are generally applied equally to both the top and bottom laminations.

Axial Members

Glulam axial combinations were developed to provide the most efficient and economical section for resisting axial forces and flexural loads applied parallel to the wide faces of the laminations. Members having loads applied parallel to the wide faces of the laminations are commonly referred to as vertically laminated members. Unlike the practice for bending combinations, the same grade of lamination is used throughout the axial combination. Axial combinations may also be loaded perpendicular to the wide face of the laminations, but the nonselective placement of material often results in a less efficient and less economical member than does the bending combination. As with bending combinations, knot and slope-of-grain requirements apply based on the intended use of the axial member as a tension or compression member.

Curved Members

Efficient use of lumber in cross sections of curved glulam combinations is similar to that in cross sections of straight, horizontally-laminated combinations. Tension and compression stresses are analyzed as tangential stresses in the curved portion of the member. A unique behavior in these curved members is the formation of radial stresses perpendicular to the wide faces of the laminations. As the radius of curvature of the glulam member decreases, the radial stresses formed in the curved portion of the beam increase. Because of the relatively low strength of lumber in tension perpendicular-to-the-grain compared with tension parallel-to-the-grain, these radial stresses become a critical factor in designing curved glulam combinations. Curved members are commonly manufactured with standard 19- and 38-mm- (nominal 1- and 2-in.-) thick lumber. Naturally, the curvature that is obtainable with the standard 19-mm- (nominal 1-in.-) thick lumber will be sharper than that for the standard 38-mm- (nominal 2-in.-) thick lumber. Recommended practice specifies that the ratio of lamination thickness t to the radius of curvature R should not exceed 1/100 for hardwoods and Southern Pine and 1/125 for other softwoods (AF&PA 1997). For example, a curved Southern Pine beam ($t/R \leq 1/100$) manufactured with standard 38-mm- (nominal 2-in.-) thick lumber (t = 1.5 in.) should have a radius of curvature greater than or equal to 3.81 m (150 in.)

Tapered Straight Members

Glulam beams are often tapered to meet architectural requirements, provide pitched roofs, facilitate drainage, and lower wall height requirements at the end supports. The taper is achieved by sawing the member across one or more laminations at the desired slope. It is recommended that the taper cut be made only on the compression side of the glulam member, because violating the continuity of the tension-side laminations would decrease the overall strength of the member. Common forms of straight, tapered glulam combinations include (a) single tapered, a member having a continuous slope from end to end on the compression side; (b) double tapered, a member having two separate slopes sawn on the compression side; (c) tapered at both ends, a member with slopes sawn on the ends, but the middle portion remains straight; and (d) tapered at one end, similar to (c) with only one end having a slope. These four examples are illustrated in Figure 11–2.

Standards and Specifications

Manufacture

The ANSI/AITC A190.1 standard of the American National Standards Institute (ANSI 1992) contains requirements for the production, testing, and certification of structural glulam timber in the United States. Additional details and commentary on the requirements specified in ANSI A190.1 are provided in AITC 200 (AITC 1993a), which is part of ANSI A190.1 by reference. A standard for glulam poles, ANSI O5.2 (ANSI 1996), addresses special requirements for utility uses. Requirements for the manufacture of structural glulam in Canada are given in CAN/CSA O122 (CSA 1989).

Derivation of Design Values

ASTM D3737 (ASTM 1997b) covers the procedures to establish design values for structural glulam timber. Properties considered include bending, tension, compression parallel to grain, modulus of elasticity, horizontal shear, radial tension, and compression perpendicular to grain.

Design Values and Procedures

Manufacturers of glulam timber have standardized the target design values in bending for beams. For softwoods, these design values are given in AITC 117, “Standard Specifications for Structural Glued-Laminated Timber of Softwood Species” (AITC 1993b). This specification contains design values and recommended modification of stresses for the design of glulam timber members in the United States. A comparable specification for hardwoods is AITC 119, “Standard Specifications for Structural Glued-Laminated Timber of Hardwood Species” (AITC 1996). The *National Design Specification for Wood Construction* (NDS) summarizes the design information in AITC 117 and 119 and defines the practice to be followed in structural design of glulam timbers (AF&PA 1997). For additional design information, see the *Timber Construction Manual* (AITC 1994). APA—The Engineered Wood Association has also developed design values for glulam under National Evaluation Report 486, which is recognized by all the model building codes.

In Canada, CAN/CSA O86, the code for engineering design in wood, provides design criteria for structural glulam timbers (CSA 1994).

Manufacture

The manufacture of glulam timber must follow recognized national standards to justify the specified engineering design values. When glulam is properly manufactured, both the quality of the wood and the adhesive bonds should demonstrate a balance in structural performance.

The ANSI A190.1 standard (ANSI 1992) has a two-phase approach to all phases of manufacturing. First is the qualification phase in which all equipment and personnel critical to the production of a quality product are thoroughly examined by a third-party agency and the strength of samples of glued joints is determined. In the second phase, after successful qualification, daily quality assurance procedures and criteria are established, which are targeted to keep each of the critical phases of the process under control. An employee is assigned responsibility for supervising the daily testing and inspection. The third-party agency makes unannounced visits to the plants to monitor the manufacturing process and the finished product and to examine the daily records of the quality assurance testing.

The manufacturing process can be divided into four major parts: (a) drying and grading the lumber, (b) end jointing the lumber, (c) face bonding, and (d) finishing and fabrication.

In instances where the glulam will be used in high moisture content conditions, it is also necessary to pressure treat the member with preservative. A final critical step in ensuring the quality of glulam is protection of the glulam timber during transit and storage.

Lumber Drying and Grading

To minimize dimensional changes following manufacture and to take advantage of the increased structural properties assigned to lumber compared with large sawn timbers, it is critical that the lumber be properly dried. This generally means kiln drying. For most applications, the maximum moisture content permitted in the ANSI standard is 16% (ANSI 1992). Also, the maximum range in moisture content is 5% among laminations to minimize differential changes in dimension following manufacture. Many plants use lumber at or slightly below 12% moisture content for two reasons. One reason is that the material is more easily end jointed at 12% moisture content than at higher levels. The other reason is that 12% is an overall average equilibrium moisture content for many interior applications in the United States (see Ch. 12, Tables 12–1 and 12–2). Exceptions are some areas in the southwest United States. Matching the moisture content of the glulam timber at the time of manufacture to that which it will attain in application minimizes shrinkage and swelling, which are the causes of checking.

The moisture content of lumber can be determined by sampling from the lumber supply and using a moisture meter. Alternatively, most manufacturers use a continuous in-line moisture meter to check the moisture content of each piece of lumber as it enters the manufacturing process. Pieces with greater than a given moisture level are removed and redried.

Grading standards published by the regional lumber grading associations describe the characteristics that are permitted in various grades of lumber. Manufacturing standards for glulam timber describe the combination of lumber grades necessary for specific design values (AITC 117) (AITC 1993b). Two types of lumber grading are used for laminating: visual grading and E-rating.

The rules for visually graded lumber are based entirely upon the characteristics that are readily apparent. The lumber grade description consists of limiting characteristics for knot sizes, slope of grain, wane, and several other characteristics. An example of the knot size limitation for visually graded western species is as follows:

Laminating grade	Maximum knot size
L1	1/4 of width
L2	1/3 of width
L3	1/2 of width

E-rated lumber is graded by a combination of lumber stiffness determination and visual characteristics. Each piece of lumber is evaluated for stiffness by one of several acceptable procedures, and those pieces that qualify for a specific grade are then visually inspected to ensure that they meet the requirement for maximum allowable edge knot size. The grades are expressed in terms of their modulus of elasticity followed by their limiting edge knot size. Thus, a 2.0E–1/6 grade has a modulus of elasticity of 13.8 GPa (2×10^6 lb/in^2) and a maximum edge knot size of 1/6 the width.

Manufacturers generally purchase graded lumber and verify the grades through visual inspection of each piece and, if E-rated, testing of a sample. To qualify the material for some of the higher design stresses for glulam timber, manufacturers must also conduct additional grading for material to be used in the tension zone of certain beams. High quality material is required for the outer 5% of the beam on the tension size, and the grading criteria for these "tension laminations" are given in AITC 117 (AITC 1993b). Special criteria are applied to provide material of high tensile strength. Another option is to purchase special lumber that is manufactured under a quality assurance system to provide the required tensile strength. Another option practiced by at least one manufacturer has been to use LVL to provide the required tensile strength.

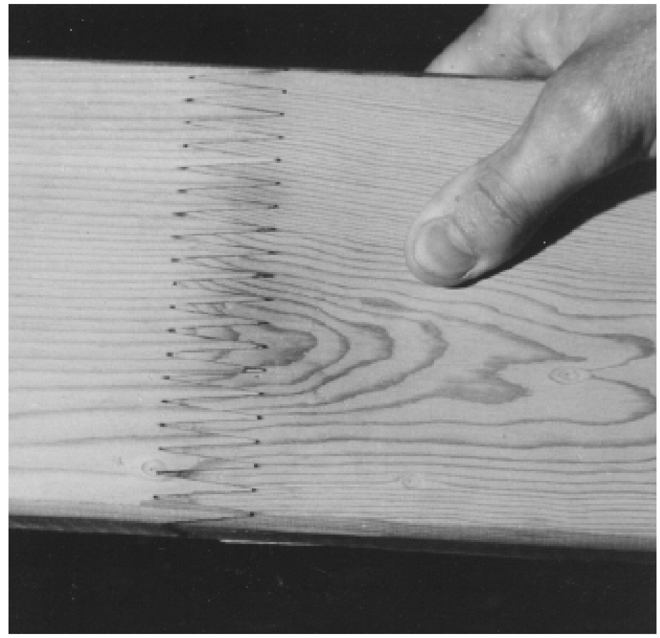

Figure 11–4. Typical fingerjoint used in the manufacture of glulam.

End Jointing

To manufacture glulam timber in lengths beyond those commonly available for lumber, laminations must be made by end jointing lumber to the proper length. The most common end joint, a fingerjoint, is about 28 mm (1.1 in.) long (Fig. 11–4). Other configurations are also acceptable, provided they meet specific strength and durability requirements. The advantages of fingerjoints are that they require only a short length of lumber to manufacture (thus reducing waste) and continuous production equipment is readily available. Well-made joints are critical to ensure adequate performance of glulam timber. Careful control at each stage of the process—determining lumber quality, cutting the joint, applying the adhesive, mating, applying end pressure, and curing—is necessary to produce consistent high strength joints.

Just prior to manufacture, the ends of the lumber are inspected to ensure that there are no knots or other features that would impair joint strength. Then, joints are cut on both ends of the lumber with special knives. Adhesive is applied. The joints in adjacent pieces of lumber are mated, and the adhesive is cured under end pressure. Most manufacturing equipment features a continuous radio-frequency curing system that provides heat to partially set the adhesive in a matter of a few seconds. Fingerjoints obtain most of their strength during this process, and residual heat permits the joint to reach its full strength within a few hours.

Fingerjoints have the potential to reach at least 75% of the strength of clear wood in many species if properly manufactured. These joints are adequate for most applications because most lumber grades used in the manufacture of glulam timber permit natural characteristics that result in strength reductions of at least 25% less than that of clear wood.

The ANSI standard requires that manufacturers qualify their production joints to meet the required strength level of the highest grade glulam timber they wish to produce. This requires that the results of tensile tests of end-jointed lumber meet certain strength criteria and that durability meets certain criteria. When these criteria are met, daily quality control testing in tension is required to ensure that the strength level is being maintained. Durability tests are also required.

A continuing challenge in the glulam production process is to eliminate the occurrence of an occasional low-strength end joint. Visual inspection and other nondestructive techniques have been shown to be only partially effective in detecting low-strength joints. An approach used by many manufacturers to ensure end joint quality is the use of a proof loading system for critical end joints. This equipment applies a specified bending or tension load to check the joint strength for critical laminations on the tension side of beams. By applying loads that are related to the strength desired, low-strength joints can be detected and eliminated. The qualification procedures for this equipment must prove that the applied loads do not cause damage to laminations that are accepted.

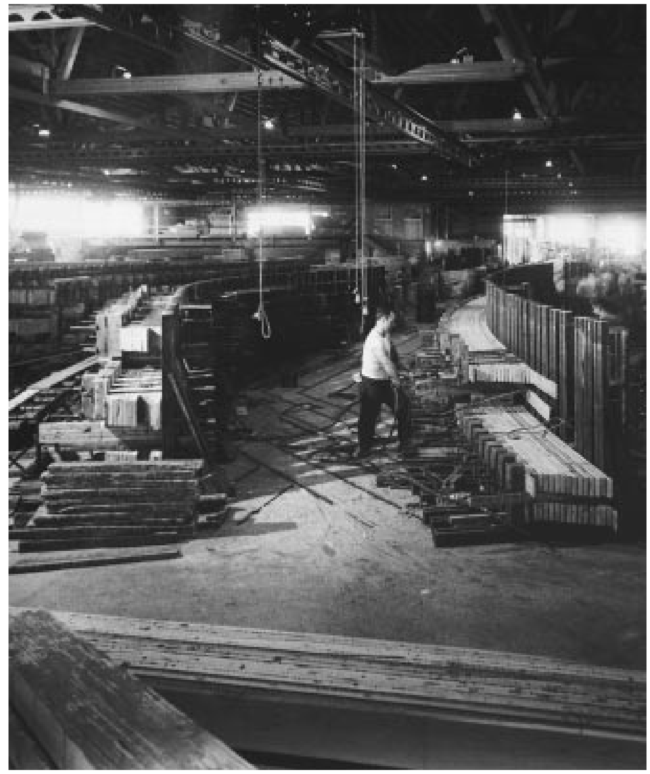

Figure 11–5. After being placed in the clamping bed, the laminations of these arches are forced together with an air-driven screw clamp.

Face Bonding

The assembly of laminations into full-depth members is another critical stage in manufacture. To obtain clear, parallel, and gluable surfaces, laminations must be planed to strict tolerances. The best procedure is to plane the two wide faces of the laminations just prior to the gluing process. This ensures that the final assembly will be rectangular and that the pressure will be applied evenly. Adhesives that have been prequalified are then spread, usually with a glue extruder. Phenol resorcinol is the most commonly used adhesive for face gluing, but other adhesives that have been adequately evaluated and proven to meet performance and durability requirements may also be used.

The laminations are then assembled into the required layup; after the adhesive is given the proper open assembly time, pressure is applied. The most common method for applying pressure is with clamping beds; the pressure is applied with either a mechanical or hydraulic system (Fig. 11–5). This results in a batch-type process, and the adhesive is allowed to cure at room temperature from 6 to 24 h. Some newer automated clamping systems include continuous hydraulic presses and radio-frequency curing to shorten the face gluing process from hours to minutes. Upon completion of the face bonding process, the adhesive is expected to have attained 90% or more of its bond strength. During the next few days, curing continues, but at a much slower rate.

The face bonding process is monitored by controls in the lumber planing, adhesive mixing, and adhesive spreading and clamping processes. Performance is evaluated by conducting shear tests on samples cut off as end trim from the finished glulam timber. The target shear strength of small specimens is prescribed in ANSI A190.1 (ANSI 1992) and equals about 90% of the average shear strength for the species. Thus, the adhesive bonds are expected to develop nearly the full strength of the wood soon after manufacture.

Finishing and Fabrication

After the glulam timber is removed from the clamping system, the wide faces are planed to remove the adhesive that has squeezed out between adjacent laminations and to smooth out any slight irregularities between the edges of adjacent laminations. As a result, the finished glulam timber is slightly narrower than nominal dimension lumber. The remaining two faces of the member can be lightly planed or sanded using portable equipment.

The appearance requirements of the beam dictate the additional finishing necessary at this point. Historically, three classifications of finishing have been included in the industry standard, AITC 110: Industrial, Architectural, and Premium (AITC 1984). Industrial appearance is generally applicable when appearance is not a primary concern, such as industrial plants and warehouses. Architectural appearance is suitable for most applications where appearance is an important requirement. Premium appearance is the highest classification. The primary difference among these classifications is the amount of knot holes and occasional planer skips that are permitted. A recently introduced classification, called Framing, consists of hit-and-miss planing and permits a significant amount of adhesive to remain on the surface. This finishing is intended for uses that require one member to have the same width as the lumber used in manufacture for framing into walls. These members are often covered in the finished structure.

The next step in the manufacturing process is fabrication, where the final cuts are made, holes are drilled, connectors are added, and a finish or sealer is applied, if specified. For various members, different degrees of prefabrication are done at this point. Trusses may be partially or fully assembled. Moment splices can be fully fabricated, then disconnected for transportation and erection. End sealers, surface sealers, primer coats, and wrapping with waterproof paper or plastic all help to stabilize the moisture content of the glulam timber between the time it is manufactured and installed. The extent of protection necessary depends upon the end use and must be specified.

Preservative Treatment

In instances where the moisture content of the finished glulam timber will approach or exceed 20% (in most exterior and some interior uses), the glulam timber should be preservative treated following AITC (1990) and AWPA (1997b). Three main types of preservatives are available: creosote, oilborne, and waterborne. Creosote and oilborne preservatives are applied to the finished glulam timbers. Some light oil solvent treatments can be applied to the lumber prior to gluing, but the suitability must be verified with the manufacturer. Waterborne preservatives are best applied to the lumber prior to the laminating and manufacturing process because they can lead to excessive checking if applied to large finished glulam timbers.

Creosote Solutions—Treatment with creosote solutions is suitable for the most severe outdoor exposure. It results in a dark, oily surface appearance that is difficult to alter. This, coupled with a distinct odor, restricts creosote solutions to structures, such as bridges, that do not come in direct contact with humans. Creosote solutions are an extremely effective preservative as proven by their continued use for railway structures. Another advantage is that the creosote treatment renders the timbers much less susceptible to moisture content changes than are untreated timbers. Creosote solutions are often used as a preservative treatment on bridge stringers.

Oilborne Treatments—Pentachlorophenol and copper napthanate are the most common oilborne preservatives. The solvents are classified in AWPA Standard P9 as Type A, Type C, and Type D (AWPA 1997a). Type A results in an oily finish and should not be used when a plain table surface is needed. Type B or C can be stained or painted. More details are given in AITC (1990) and AWPA (1997a).

Waterborne Treatments—Waterborne preservative treatments conform to AWPA P5 (AWPA 1997b) and use water-soluble preservative chemicals that become fixed in the wood. The effectiveness of this treatment depends upon the depth to which the chemicals penetrate into the lumber. Different processes are quite effective for some species but not for others. In addition, the treated lumber is generally more difficult to bond effectively and requires special manufacturing procedures. Thus, it is recommended that the manufacturer be contacted to determine the capabilities of waterborne-preservative-treated products.

The major advantage of a waterborne treatment is that the surface of the timber appears little changed by the treatment. Different chemicals can leave a green, gray, or brown color; all result in a surface that is easily finished with stains or paints. To avoid the potential of corrosive interactions with the chemical treatments, special care must be given when selecting the connection hardware. In addition, waterborne-preservative-treated glulam timber is much more subject to moisture content cycling than is creosote-treated or oilborne-preservative-treated glulam timber.

A major consideration in selecting a preservative treatment is the local regulations dealing with the use and disposal of waste from preservative-treated timber. Recommended retention levels for applications of various preservatives are given in AITC 109 (AITC 1990) along with appropriate quality assurance procedures.

Development of Design Values

The basic approach to determine the engineered design values of glulam members is through the use of stress index values and stress modification factors.

Stress Index Values

Stress index values are related to the properties clear of wood that is free of defects and other strength-reducing characteristics. Stress index values for several commonly used species and E-rated grades of lumber are given in ASTM D3737 (ASTM 1997b). Procedures are also given for developing these values for visual grades of other species.

Stress Modification Factors

Stress modification factors are related to strength-reducing characteristics and are multiplied by the stress index values to obtain allowable design properties. Detailed information on determination of these factors for bending, tension, compression, and modulus of elasticity are given in ASTM D3737 (1997b).

Other Considerations

Effect of End Joints on Strength—Both fingerjoints and scarf joints can be manufactured with adequate strength for use in structural glulam. Adequacy is determined by physical testing procedures and requirements in ANSI A190.1 (ANSI 1992).

Joints should be well scattered in portions of structural glulam that is highly stressed in tension. Required spacings of end joints are given in ANSI A190.1. End joints of two qualities can be used in a glulam member, depending upon strength requirements at various depths of the cross section.

However, laminators usually use the same joint throughout the members for ease in manufacture.

The highest strength values are obtained with well-made plain scarf joints; the lowest values are obtained with butt joints. This is because scarf joints with flat slopes have essentially side-grain surfaces that can be well bonded to develop high strength, and butt joints have end-grain surfaces that cannot be bonded effectively. Structural fingerjoints (either vertical or horizontal) are a compromise between scarf and butt joints; the strength of structural fingerjoints varies with joint design.

No statement can be made regarding the specific joint strength factor of fingerjoints, because fingerjoint strength depends on the type and configuration of the joint and the manufacturing process. However, the joint factor of commonly used fingerjoints in high-quality lumber used for laminating can be about 75%. High-strength fingerjoints can be made when the design is such that the fingers have relatively flat slopes and sharp tips. Tips are essentially a series of butt joints that reduce the effectiveness of fingerjoints as well as creating sources of stress concentration.

Generally, butt joints cannot transmit tensile stress and can transmit compressive stress only after considerable deformation or if a metal bearing plate is tightly fitted between the abutting ends. In normal assembly operations, such fitting would not be done. Therefore, it is necessary to assume that butt joints are ineffective in transmitting both tensile and compressive stresses. Because of this ineffectiveness and because butt joints cause concentration of both shear stress and longitudinal stress, butt joints are not permitted for use in structural glued-laminated timbers.

Effect of Edge Joints on Strength—It is sometimes necessary to place laminations edge-to-edge to provide glulam members of sufficient width. Because of difficulties in fabrication, structural edge joint bonding may not be readily available, and the designer should investigate the availability of such bonding prior to specifying.

For tension, compression, and horizontally laminated bending members, the strength of edge joints is of little importance to the overall strength of the member. Therefore, from the standpoint of strength, it is unnecessary that edge joints be glued if they are not in the same location in adjacent laminations. However, for maximum strength, edge joints should be glued where torsional loading is involved. Other considerations, such as the appearance of face laminations or the possibility that water will enter the unglued joints and promote decay, should also dictate if edge joints are glued.

If edge joints in vertically laminated beams are not glued, shear strength could be reduced. The amount of reduction can be determined by engineering analysis. Using standard laminating procedures with edge joints staggered in adjacent laminations by at least one lamination thickness, shear strength of vertically laminated beams with unglued edge joints is approximately half that of beams with adhesive-bonded edge joints.

Effect of Shake, Checks, and Splits on Shear Strength—In general, checks and splits have little effect on the shear strength of glulam. Shake occurs infrequently and should be excluded from material for laminations. Most laminated timbers are made from laminations that are thin enough to season readily without developing significant checks and splits.

Designs for Glued-Laminated Timber

Most basic engineering equations used for sawn lumber also apply to glulam beams and columns. The design of glulam in this chapter is only applicable to glulam combinations that conform to AITC 117 (AITC 1993b) for softwood species and AITC 119 (AITC 1996) for hardwood species and are manufactured in accordance with ANSI/AITC A190.1 (ANSI 1992). The AITC 117 standard is made up of two parts: (a) manufacturing, which provides details for the many configurations of glulam made from visually graded and *E*-rated softwood lumber; and (b) design, which provides tabular design values of strength and stiffness for these glulam combinations. The AITC 119 standard provides similar information for glulam made from hardwood species of lumber. These standards are based on laterally-braced straight members with an average moisture content of 12%. For bending members, the design values are based on an assumed reference size of 305 mm deep, 130 mm wide, and 6.4 m long (12 in. deep, 5.125 in. wide, and 21 ft long).

Tabular Design Values

Tabular design values given in AITC 117 and AITC 119 include the following:

F_b allowable bending design value,

F_t allowable tension design value parallel to grain,

F_v allowable shear design value parallel to grain,

$F_{c\text{-perp}}$ allowable compression design value perpendicular to grain,

F_c allowable compression design value parallel to grain,

E allowable modulus of elasticity, and

F_{rt} allowable radial tension design value perpendicular to grain.

Because glulam members can have different properties when loaded perpendicular or parallel to the wide faces of the laminations, a common naming convention is used to specify the design values that correspond to a particular type of orientation. For glulam members loaded perpendicular to the wide faces of the laminations, design values are commonly denoted with a subscript x. For glulam members loaded parallel to the wide faces of the laminations, design values are commonly denoted with a subscript y. Some examples include F_{bx} and E_x for design bending stress and design modulus of elasticity, respectively.

End-Use Adjustment Factors

When glulam members are exposed to conditions other than the described reference condition, the published allowable design values require adjustment. The following text describes each of the adjustment factors that account for the end-use condition of glulam members.

Volume—The volume factor C_v accounts for an observed reduction in strength when length, width, and depth of structural glulam members increase. This strength reduction is due to the higher probability of occurrence of strength-reducing characteristics, such as knots and slope of grain, in higher volume beams. This volume factor adjustment is given in the *National Design Specification* for *Wood Construction* (AF&PA 1997) in the form

$$C_v = \left(\frac{305}{d}\right)^{0.10}\left(\frac{130}{w}\right)^{0.10}\left(\frac{6.4}{L}\right)^{0.10} \quad \text{(metric)} \qquad (11\text{–}1a)$$

$$C_v = \left(\frac{12}{d}\right)^{0.10}\left(\frac{5.125}{w}\right)^{0.10}\left(\frac{21}{L}\right)^{0.10} \quad \text{(inch– pound)} \qquad (11\text{–}1b)$$

for Douglas-fir and other species, and

$$C_v = \left(\frac{305}{d}\right)^{0.05}\left(\frac{130}{w}\right)^{0.05}\left(\frac{6.4}{L}\right)^{0.05} \quad \text{(metric)} \qquad (11\text{–}2a)$$

$$C_v = \left(\frac{12}{d}\right)^{0.05}\left(\frac{5.125}{w}\right)^{0.05}\left(\frac{21}{L}\right)^{0.05} \quad \text{(inch–pound)} \qquad (11\text{–}2b)$$

for southern pines, where d is depth (mm, in.), w width (mm, in.), and L length (m, ft). (Eqs. (11–1a) and (11–2a) in metric, Eqs. (11–1b) and (11–2b) in inch–pound system.)

Moisture Content—The moisture content factor C_M accounts for the reduction in strength as moisture content increases. A moisture content adjustment is listed in both ASTM D3737 (ASTM 1997b) and AITC 117–Design (AITC 1993b).

C_M = 1.0 for moisture content ≤16%

For moisture content >16%, as in ground contact and many other exterior conditions, use the following C_M values:

	F_b	F_t	F_v	$F_{c\text{-perp}}$	F_c	E
C_M	0.8	0.8	0.875	0.53	0.73	0.833

Loading—An adjustment for the type of loading on the member is also necessary because the volume factors are derived assuming a uniform load. This method of loading factor C_L is recommended in the *National Design Specification for Wood Construction* (AF&PA 1997).

C_L = 1.00 for uniform loading on a simple span

= 1.08 for center point loading on a simple span

= 0.92 for constant stress over the full length

For other loading conditions, values of C_L can be estimated using the proportion of the beam length subjected to 80% or more of the maximum stress L_0 and

$$C_L = \left(\frac{0.45}{L_0}\right)^{0.1} \qquad (11\text{–}3)$$

Tension Lamination—Past research has shown that special provisions are required for the tension lamination of a glulam beam to achieve the specified design bending strength levels. Properties listed in AITC 117 and 119 are applicable to beams with these special tension laminations. If a special tension lamination is not included in the beam combination, strength reduction factors must be applied. Tension lamination factors C_T, which can be found in ASTM D3737 (ASTM 1997b), have the following values:

C_T = 1.00 for special tension laminations per AITC 117

= 0.85 without tension laminations and for depth ≤380 mm (≤15 in.)

= 0.75 without tension laminations and for depth >380 mm (>15 in.).

Curvature—The curvature factor accounts for the increased stresses in the curved portion of curved glulam beams. This factor does not apply to design values in the straight portion of a member, regardless of the curvature elsewhere. The curvature factor C_c, which can be found in the *National Design Specification* (AF&PA 1997), has the following relation:

$$C_c = 1 - 2000\left(\frac{t}{R}\right)^2 \qquad (11\text{–}4)$$

where t is thickness of lamination and R is radius of curvature on inside face of lamination. The value $t/R \le 1/100$ for hardwoods and southern pines; $t/R \le 1/125$ for other softwoods.

Flat Use—The flat use factor is applied to bending design values when members are loaded parallel to wide faces of laminations and are less than 305 mm (12 in.) in depth. Flat use factors C_{fu}, which can be found in the *National Design Specification* (AF&PA 1997), have the following values:

Member dimension parallel to wide faces of laminations	C_{fu}
273 or 267 mm (10-3/4 or 10-1/2 in.)	1.01
222 or 216 mm (8-3/4 or 8-1/2 in.)	1.04
171 mm (6-3/4 in.)	1.07
130 or 127 mm (5-1/8 or 5 in.)	1.10
79 or 76 mm (3-1/8 or 3 in.)	1.16
64 mm (2-1/2 in.)	1.19

Lateral Stability—The lateral stability factor is applied to bending design values to account for the amount of lateral support applied to bending members. Deep bending members that are unsupported along the top surface are subject to lateral torsional buckling and would have lower bending design values. Members that are fully supported would have no adjustments (C_L = 1.0).

Glued Members With Lumber and Panels

Highly efficient structural components can be produced by combining lumber with panel products through gluing. These components, including box beams, I-beams, "stressed-skin" panels, and folded plate roofs, are discussed in detail in technical publications of the APA—The Engineered Wood Association (APA 1980). One type of member, prefabricated wood I-joists, is discussed in detail. Details on structural design are given in the following portion of this chapter for beams with webs of structural panel products and stressed-skin panels wherein the parts are glued together with a rigid, durable adhesive.

These highly efficient designs, although adequate structurally, can suffer from lack of resistance to fire and decay unless treatment or protection is provided. The rather thin portions of the cross section (the panel materials) are more vulnerable to fire damage than are the larger, solid cross sections.

Box Beams and I-Beams

Box beams and I-beams with lumber or laminated flanges and structural panel webs can be designed to provide the desired stiffness, bending, moment resistance, and shear resistance. The flanges resist bending moment, and the webs provide primary shear resistance. Proper design requires that the webs must not buckle under design loads. If lateral stability is a problem, the box beam design should be chosen because it is stiffer in lateral bending and torsion than is the I-beam. In contrast, the I-beam should be chosen if buckling of the web is of concern because its single web, double the thickness of that of a box beam, will offer greater buckling resistance.

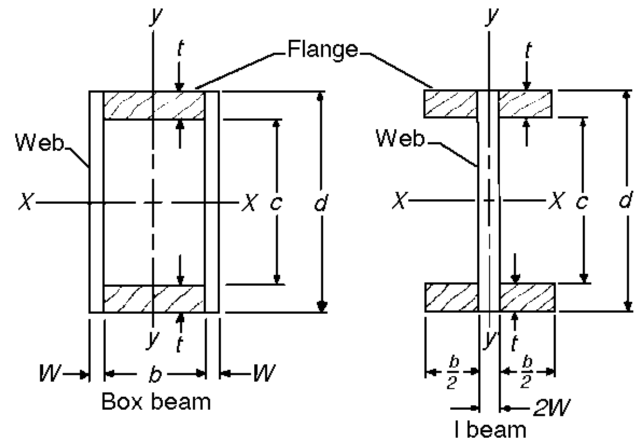

Figure 11–6. Beams with structural panel webs.

Design details for beam cross sections (including definitions of terms in the following equations) are presented in Figure 11–6. Both flanges in these beams are the same thickness because a construction symmetrical about the neutral plane provides the greatest moment of inertia for the amount of material used. The following equations were derived by basic principles of engineering mechanics. These methods can be extended to derive designs for unsymmetrical constructions, if necessary.

Beam Deflections

Beam deflections can be computed using Equation (8–2) in Chapter 8. The following equations for bending stiffness $(EI)_x$ and shear stiffness GA' apply to the box and I-beam shown in Figure 11–6. The bending stiffness is given by

$$(EI)_x = \frac{1}{12}[E(d^3 - c^3)b + 2E_w W d^3] \qquad (11\text{–}5)$$

where E is flange modulus of elasticity and E_w is web modulus of elasticity. For plywood, values of E_w for the appropriate structural panel construction and grain direction can be computed from Equations (11–1), (11–2), and (11–3).

An approximate expression for the shear stiffness is

$$GA' = 2WcG \qquad (11\text{–}6)$$

where G is shear modulus for the structural panel for appropriate direction and A' is the effective area of the web. An improvement in shear stiffness can be made by properly orienting the web, depending upon its directional properties. Equation (11–6) is conservative because it ignores the shear stiffness of the flange. This contribution can be included by use of APA design methods that are based on Orosz (1970). (For further information on APA design methods, contact APA—The Engineered Wood Association in Tacoma, Washington.)

Flange Stresses

Flange compressive and tensile stresses at outer beam fibers are given by

$$f_x = \frac{6M}{(d^3 - c^3)\dfrac{b}{d} + \dfrac{2E_w Wd^2}{E}} \tag{11–7}$$

where M is bending moment.

Web Shear Stress

Web shear stress at the beam neutral plane is given by

$$f_{xy} = \frac{3V}{4W}\left[\frac{E(d^2 - c^2)b + 2E_w Wd^2}{E(d^3 - c^3)b + 2E_w Wd^3}\right] \tag{11–8}$$

where V is shear load. The shear stress must not exceed allowable values. To avoid web buckling, either the web should be increased in thickness or the clear length of the web should be broken by stiffeners glued to the web.

Web edgewise bending stresses at the inside of the flanges can be computed by

$$f_{xw} = \frac{6M}{\dfrac{E}{E_w}(d^3 - c^3)\dfrac{b}{c} + 2\dfrac{d^3}{c}W} \tag{11–9}$$

Although it is not likely, the web can buckle as a result of bending stresses. Should buckling as a result of edgewise bending appear possible, the interaction of shear and edgewise bending buckling can be examined using the principles of Timoshenko (1961).

Lateral Buckling

Possible lateral buckling of the entire beam should be checked by calculating the critical bending stress (Ch. 8, Lateral–Torsional Buckling section). The slenderness factor p, required to calculate this stress, includes terms for lateral flexural rigidity EI_y and torsional rigidity GK that are defined as follows:

For box beams,

$$EI_y = \frac{1}{12} E(d - c)b^3 + E_w[(b + 2W)^3 - b^3]d \tag{11–10}$$

$$GK = \left[\frac{(d + c)(d^2 - c^2)(b + W)^2 W}{(d^2 - c^2) + 4(b + W)W}\right]G \tag{11–11}$$

For I-beams,

$$EI_y = \frac{1}{12}\left\{E[(b + 2W)^3 - (2W)^3](d - c) + E_{fw}(2W)^3 d\right\} \tag{11–12}$$

$$GK = \frac{1}{3}\left[\frac{1}{4}(d - c)^3 b + d(2W)^3\right]G \tag{11–13}$$

where E_{fw} is flexural elastic modulus of the web.

In Equations (11–11) and (11–13), the shear modulus G can be assumed without great error to be about 1/16 of the flange modulus of elasticity E_L. The resultant torsional stiffness GK will be slightly low if beam webs have plywood grain at 45° to the neutral axis. The lateral buckling of I-beams will also be slightly conservative because bending rigidity of the flange has been neglected in writing the equations given here. If buckling of the I-beam seems possible at design loads, the more accurate analysis of Forest Products Laboratory Report 1318B (Lewis and others 1943) should be used before redesigning.

Stiffeners and Load Blocks

Determination of the number and sizes of stiffeners and load blocks needed in a particular construction does not lend itself to a rational procedure, but certain general rules can be given that will help the designer of a structure obtain a satisfactory structural member. Stiffeners serve a dual purpose in a structural member of this type. One function is to limit the size of the unsupported panel in the web, and the other is to restrain the flanges from moving toward each other as the beam is stressed.

Stiffeners should be glued to the webs and in contact with both flanges. A rational way of determining how thick the stiffener should be is not available, but tests of box beams made at the Forest Products Laboratory indicate that a thickness of at least six times the thickness of the web is sufficient. Because stiffeners must also resist the tendency of the flanges to move toward each other, the stiffeners should be as wide as (extend to the edge of) the flanges.

For plywood webs containing plies with the grain of the wood oriented both parallel and perpendicular to the axis of the member, the spacing of the stiffeners is relatively unimportant for the web shear stresses that are allowed. Maximum allowable stresses are less than those that will produce buckling. A clear distance between stiffeners equal to or less than two times the clear distance between flanges is adequate. Load blocks are special stiffeners placed along the member at points of concentrated load. Load blocks should be designed so that stresses caused by a load that bears against the side-grain material in the flanges do not exceed the allowable design for the flange material in compression perpendicular to grain.

Prefabricated Wood I-Joists

In recent years, the development of improved adhesives and manufacturing techniques has led to the development of the prefabricated I-joist industry. This product is a unique type of I-beam that is replacing wider lumber sizes in floor and roof applications for both residential and commercial buildings (Fig. 11–7).

Significant savings in materials are possible with prefabricated I-joists that use either plywood or oriented strandboard (OSB) for the web material and small dimension lumber or structural composite lumber (SCL) for the flanges. The high quality lumber needed for these flanges has been difficult to obtain using visual grading methods, and both mechanically

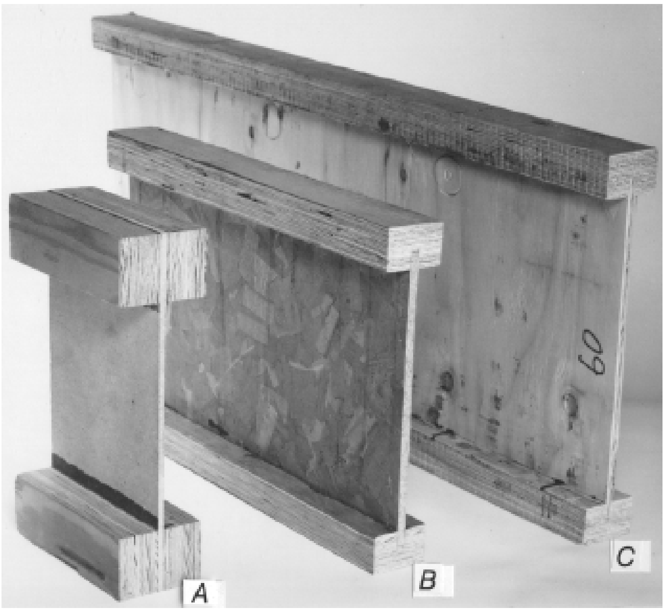

Figure 11–7. Prefabricated I-joists with laminated veneer lumber flanges and structural panel webs. (A) One experimental product has a hardboard web. The other two commercial products have (B) oriented strandboard and (C) plywood webs.

graded lumber and SCL are being used by several manufacturers. The details of fastening the flanges to the webs vary between manufacturers; all must be glued with a waterproof adhesive. Prefabricated I-joists are becoming popular with builders because of their light weight, dimensional stability and ease of construction. Their accurate and consistent dimensions, as well as uniform depth, allow the rapid creation of a level floor. Utility lines pass easily through openings in the webs.

The ASTM standard D5055 (ASTM 1997d) gives procedures for establishing, monitoring, and reevaluating structural capacities of prefabricated I-joists. Each manufacturer of prefabricated I-joists is responsible for developing the required property information and ensuring that the minimum levels of quality are maintained during production. An independent inspection agency is required to monitor the quality assurance program.

Standard grades, sizes, and span tables have not been established for all prefabricated I-joists. The production of each manufacturer may have unique design properties and procedures. Thus, the designer must consult information provided by the manufacturer. Many engineering equations presented in the previous section also apply to prefabricated I-joists.

During the 1980s, the prefabricated wood I-joists industry was one of the fastest growing segments of the wood products industry. Prefabricated I-joists are manufactured by about 15 companies in the United States and Canada and are often distributed through building material suppliers. Each manufacturer has developed its building code acceptance and provides catalogs with span tables and design information.

Recently, a performance standard for prefabricted I-joists has been promulgated for products used in residential floor construction (APA 1997).

Stressed-Skin Panels

Constructions consisting of structural panel "skins" glued to wood stringers are often called stressed-skin panels. These panels offer efficient structural constructions for floor, wall, and roof components. They can be designed to provide desired stiffness, bending moment resistance, and shear resistance. The skins resist bending moment, and the wood stringers provide shear resistance.

The details of design for a panel cross section are given in Figure 11–8. The following equations were derived by basic principles of engineering mechanics. A more rigorous design procedure that includes the effects of shear lag is available in Kuenzi and Zahn (1975).

Panel deflections can be computed using Equation (8–2) in Chapter 8. The bending stiffness EI and shear stiffness GA' are given by the following equations for the stressed-skin panel shown in Figure 11–8.

$$\begin{aligned} EI = {} & \left[\frac{b}{\left(E_1 t_1 + E_2 t_2 + E t_c (s/b)\right)} \right] \\ & \times \Big\{ E_1 t_1 E_2 t_2 [(t_1 + t_c) + (t_2 + t_c)]^2 \\ & \quad + E_1 t_1 E t_c (s/b)(t_1 + t_c)^2 + E_2 t_2 E t_c (s/b)(t_2 + t_c)^2 \Big\} \\ & + \frac{b}{12}\left(E_{f1} t_1^3 + E_{f2} t_2^3 + E t_c^3 \frac{s}{b} \right) \end{aligned} \tag{11–14}$$

where E_1 and E_2 are modulus of elasticity values for skins 1 and 2, E_{f1} and E_{f2} flexural modulus of elasticity values for skins 1 and 2, E stringer modulus of elasticity, and s total width of all stringers in a panel.

An approximate expression for shear stiffness is

$$GA' = Gst_c \tag{11–15}$$

where G is stringer shear modulus.

Skin Stresses

Skin tensile and compressive stresses are given by

$$\begin{aligned} f_{x1} &= \frac{ME_1 y_1}{EI} \\ f_{x2} &= \frac{ME_2 y_2}{EI} \end{aligned} \tag{11–16}$$

where EI is given by Equation (11–14), M is bending moment, and

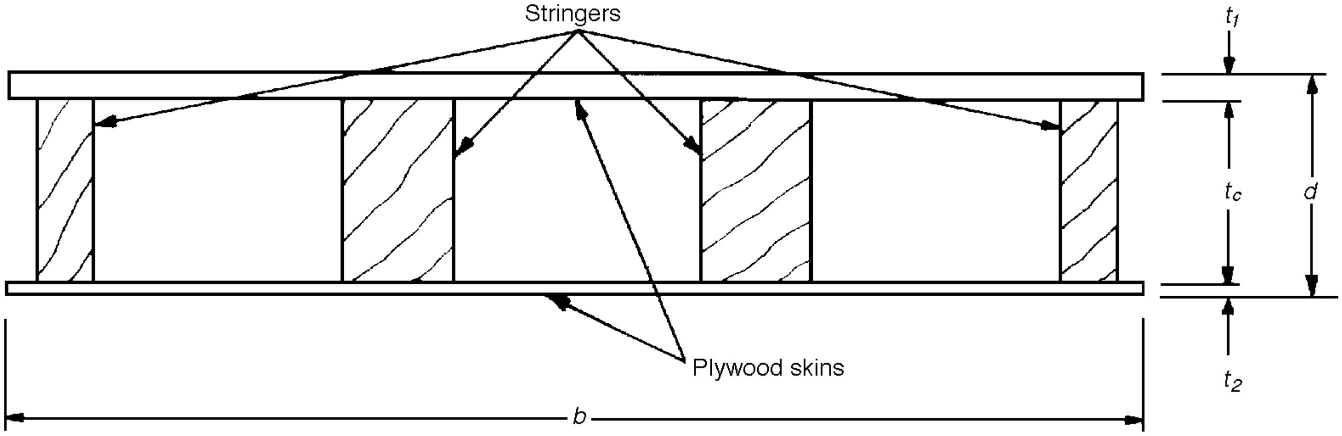

Figure 11–8. Stressed-skin panel cross section.

$$y_1 = \frac{E_2 t_2[(t_1 + t_c) + (t_2 + t_c)] + Et_c \frac{s}{b}(t_1 + t_c)}{2\left(E_1 t_1 + E_2 t_2 + Et_c \frac{s}{b}\right)}$$

$$y_2 = \frac{E_1 t_1[(t_1 + t_c) + (t_2 + t_c)] + Et_c \frac{s}{b}(t_2 + t_c)}{2\left(E_1 t_1 + E_2 t_2 + Et_c \frac{s}{b}\right)}$$

Either the skins should be thick enough or the stringers spaced closely enough so that buckling does not occur in the compression skin. Buckling stress can be analyzed by the principles in Ding and Hou (1995). The design stress for the structural panel in tension and compression strength should not be exceeded.

Stringer Bending Stress

The stringer bending stress is the larger value given by

$$f_{sx1} = \frac{ME(y_1 - t_1/2)}{EI}$$
$$f_{sx2} = \frac{ME(y_2 - t_2/2)}{EI} \qquad (11\text{–}17)$$

and these should not exceed appropriate values for the species.

The stringer shear stress is given by

$$f_{sxy} = \frac{V(EQ)}{sEI} \qquad (11\text{–}18)$$

where $EQ = (E_1 t_1 b + Es\, y_1/2)\, y_1$. This also should not exceed appropriate values for the species.

Glue Shear Stress

Glue shear stress in the joint between the skins and stringers is given by

$$f_{gl} = \frac{V(EQ)}{sEI} \qquad (11\text{–}19)$$

where $EQ = E_1 t_1 b y_1$. This stress should not exceed values for the glue and species. It should also not exceed the wood stress f_{TR} (“rolling” shear) for solid wood because, for plywood, the thin plies allow the glue shear stresses to be transmitted to adjacent plies and could cause rolling shear failure in the wood.

Buckling

Buckling of the stressed-skin panel of unsupported length under end load applied in a direction parallel to the length of the stringers can be computed by

$$P_{cr} = \frac{\pi^2 EI}{L^2} \qquad (11\text{–}20)$$

where L is unsupported panel length and EI is bending stiffness given by Equation (11–14).

Compressive stress in the skins is given by

$$f_{xc1} = \frac{PE_1}{EA}$$
$$f_{xc2} = \frac{PE_2}{EA} \qquad (11\text{–}21)$$

and in the stringers by

$$f_{sxc} = \frac{PE}{EA} \qquad (11\text{–}22)$$

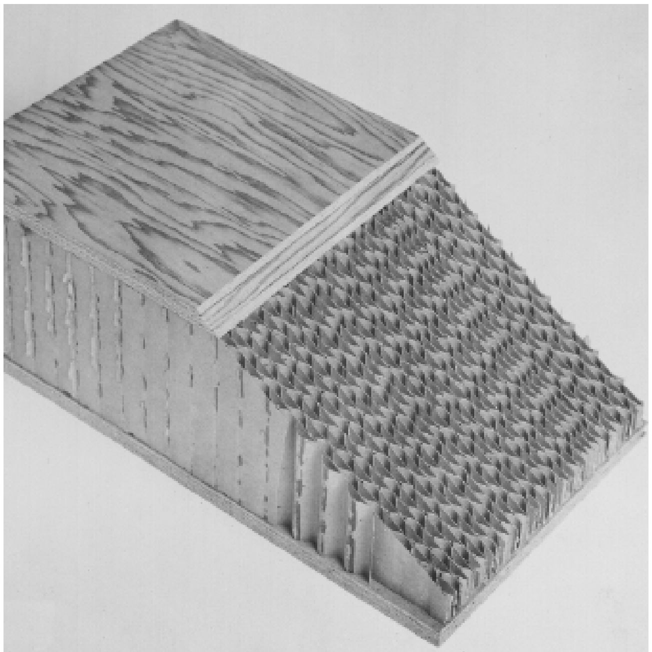

Figure 11–9. Cutaway section of sandwich construction with plywood facings and a paper honeycomb core.

where $EA = E_1t_1b + E_2t_2b + Et_cs$. These compressive stresses should not exceed stress values for the structural panel or stringer material. For plywood, compressive stress should also be less than the critical buckling stress.

Structural Sandwich Construction

Structural sandwich construction is a layered construction formed by bonding two thin facings to a thick core (Fig. 11–9). The thin facings are usually made of a strong and dense material because they resist nearly all the applied edgewise loads and flatwise bending moments. The core, which is made of a weak and low density material, separates and stabilizes the thin facings and provides most of the shear rigidity of the sandwich construction. By proper choice of materials for facings and core, constructions with high ratios of stiffness to weight can be achieved. As a crude guide to the material proportions, an efficient sandwich is obtained when the weight of the core is roughly equal to the total weight of the facings. Sandwich construction is also economical because the relatively expensive facing materials are used in much smaller quantities than are the usually inexpensive core materials. The materials are positioned so that each is used to its best advantage.

Specific nonstructural advantages can be incorporated in a sandwich construction by proper selection of facing and core materials. An impermeable facing can act as a moisture barrier for a wall or roof panel in a house; an abrasion-resistant facing can be used for the top facing of a floor panel; and decorative effects can be obtained by using panels with plastic facings for walls, doors, tables, and other furnishings. Core material can be chosen to provide thermal insulation, fire resistance, and decay resistance. Because of the light weight of structural sandwich construction, sound transmission problems must also be considered in choosing sandwich component parts.

Methods of joining sandwich panels to each other and other structures must be planned so that the joints function properly and allow for possible dimensional change as a result of temperature and moisture variations. Both structural and nonstructural advantages need to be analyzed in light of the strength and service requirements for the sandwich construction. Moisture-resistant facings, cores, and adhesives should be used if the construction is to be exposed to adverse moisture conditions. Similarly, heat-resistant or decay-resistant facings, cores, and adhesives should be used if exposure to elevated temperatures or decay organisms is expected.

Fabrication

Facing Materials

One advantage of sandwich construction is the great latitude it provides in choice of facings and the opportunity to use thin sheet materials because of the nearly continuous support by the core. The stiffness, stability, and to a large extent, the strength of the sandwich are determined by the characteristics of the facings. Facing materials include plywood, single veneers, or plywood overlaid with a resin-treated paper, oriented strandboard, hardboard, particleboard, glass–fiber-reinforced polymers or laminates, veneer bonded to metal, and metals, such as aluminum, enameled steel, stainless steel, magnesium, and titanium.

Core Materials

Many lightweight materials, such as balsa wood, rubber foam, resin-impregnated paper, reinforced plastics, perforated chipboard, expanded plastics, foamed glass, lightweight concrete and clay products, and formed sheets of cloth, metal, or paper have been used as a core for sandwich construction. New materials and new combinations of old materials are constantly being proposed and used. Cores of formed sheet materials are often called honeycomb cores. By varying the sheet material, sheet thickness, cell size, and cell shape, cores of a wide range in density can be produced. Various core configurations are shown in Figures 11–10 and 11–11. The core cell configurations shown in Figure 11–10 can be formed to moderate amounts of single curvature, but cores shown in Figure 11–11 as configurations A, B, and C can be formed to severe single curvature and mild compound curvature (spherical).

Four types of readily formable cores are shown as configurations D, E, F, and G in Figure 11–11. The type D and F cores form to a cylindrical shape, the type D and E cores to a spherical shape, and the type D and G cores to various compound curvatures.

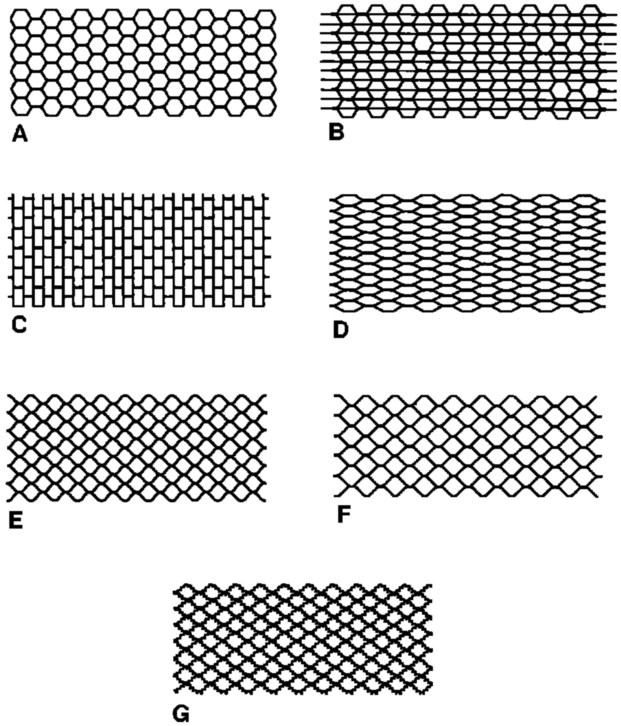

Figure 11–10. Honeycomb core cell configurations.

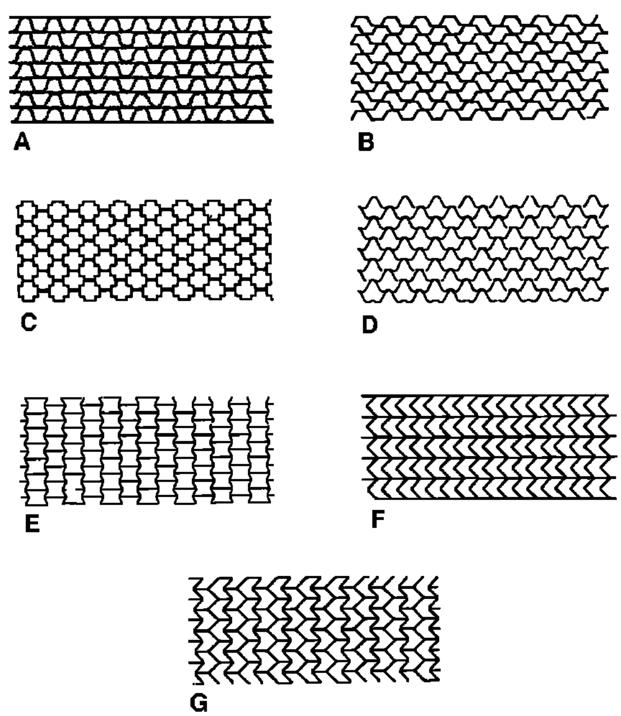

Figure 11–11. Cell configurations for formable paper honeycomb cores.

If the sandwich panels are likely to be subjected to damp or wet conditions, a core of paper honeycomb should contain a synthetic resin. When wet, paper with 15% phenolic resin provides good strength, decay resistance, and desirable handling characteristics during fabrication. Resin amounts in excess of about 15% do not seem to produce a gain in strength commensurate with the increased quantity of resin required. Smaller amounts of resin may be combined with fungicides to offer primary protection against decay.

Manufacturing Operations

The principal operation in the manufacture of sandwich panels is bonding the facings to the core. Special presses are needed for sandwich panel manufacture to avoid crushing lightweight cores, because the pressures required are usually lower than can be obtained in the range of good pressure control on presses ordinarily used for structural panels or plastic products. Because pressure requirements are low, simple and perhaps less costly presses could be used. Continuous roller presses or hydraulic pressure equipment may also be suitable. In the pressing of sandwich panels, special problems can occur, but the manufacturing process is basically not complicated.

Adhesives must be selected and applied to provide the necessary joint strength and permanence. The facing materials, especially if metallic, may need special surface treatment before the adhesive is applied.

In certain sandwich panels, loading rails or edgings are placed between the facings at the time of assembly. Special fittings or equipment, such as heating coils, plumbing, or electrical wiring conduit, can easily be installed in the panel before its components are fitted together.

Some of the most persistent difficulties in the use of sandwich panels are caused by the necessity of introducing edges, inserts, and connectors. In some cases, the problem involves tying together thin facing materials without causing severe stress concentrations. In other cases, such as furniture manufacture, the problem is “show through” of core or inserts through decorative facings. These difficulties are minimized by a choice of materials in which the rate and degree of differential dimensional movement between core and insert are at a minimum.

Structural Design

The structural design of sandwich construction can be compared with the design of an I-beam. The facings and core of the sandwich are analogous to the flanges and web of the I-beam, respectively. The two thin and stiff facings, separated by a thick and light core, carry the bending loads. The functions of the core are to support the facings against lateral wrinkling caused by in-plane compressive loads and to carry, through the bonding adhesive, shear loads. When the strength requirements for the facings and core in a particular design are met, the construction should also be checked for possible buckling, as for a column or panel in compression, and for possible wrinkling of the facings.

The contribution of the core material to the stiffness of the sandwich construction can generally be neglected because of the core's low modulus of elasticity; when that is the case, the shear stress can be assumed constant over the depth of the core. The facing moduli of elasticity are usually more than 100 times as great as the core modulus of elasticity. The core material may also have a small shear modulus. This small shear modulus causes increased deflections of sandwich construction subjected to bending and decreased buckling loads of columns and edge-loaded panels, compared with constructions in which the core shear modulus is high. The effect of this low shear modulus is greater for short beams and columns and small panels than it is for long beams and columns and large panels.

Without considering the contribution of core material, the bending stiffness of sandwich beams having facings of equal or unequal thickness is given by

$$D = \frac{h^2 t_1 t_2 (E_1 t_2 + E_2 t_1)}{(t_1 + t_2)^2} + \frac{1}{12}(E_1 t_1^3 + E_2 t_2^3) \qquad (11\text{–}23)$$

where D is the stiffness per unit width of sandwich construction (product of modulus of elasticity and moment of inertia of the cross section), E_1 and E_2 moduli of elasticity of facings 1 and 2, t_1 and t_2 facing thickness, and h distance between facing centroids.

The shear stiffness per unit width is given by

$$U = \frac{h^2}{t_c} G_c \qquad (11\text{–}24)$$

where G_c is the core shear modulus associated with distortion of the plane perpendicular to the facings and parallel to the sandwich length and t_c is the thickness of the core.

The bending stiffness D and shear stiffness U are used to compute deflections and buckling loads of sandwich beams. The general expression for the deflection of flat sandwich beams is given by

$$\frac{d^2 y}{dx^2} = -\frac{M_x}{D} + \frac{1}{U}\left(\frac{dS_x}{dx}\right) \qquad (11\text{–}25)$$

where y is deflection, x distance along the beam, M_x bending moment per unit width at point x, and S_x shear force per unit width at point x.

Integration of Equation (11–25) leads to the following general expression for deflection of a sandwich beam:

$$y = \frac{k_b P a^3}{D} + \frac{k_s P a}{U} \qquad (11\text{–}26)$$

where P is total load per unit width of beam, a is span, and k_b and k_s are constants dependent upon the loading condition. The first term in the right side of Equation (11–26) gives the bending deflection and the second term the shear deflection. Values of k_b and k_s for several loadings are given in Table 11–1.

For sandwich panels supported on all edges, the theory of plates must be applied to obtain analytical solutions. A comprehensive treatment of sandwich plates under various loading and boundary conditions can be found in the books by Allen (1969), Whitney (1987), and Vinson and Sierakowski (1986). Many extensive studies of sandwich construction performed at the Forest Products Laboratory are referenced in those books. In addition, some high-order analyses of sandwich construction that consider general material properties for component parts in specified applications can be found in the references at the end of this chapter.

The buckling load per unit width of a sandwich panel with no edge members and loaded as a simply supported column is given by

$$N = \frac{N_E}{1 + N_E/U} \qquad (11\text{–}27)$$

where critical load

$$N_E = \frac{\pi^2 n^2 D}{a^2} \qquad (11\text{–}28)$$

in which n is the number of half-waves into which the column buckles and a is the panel length. The minimum value of N_E is obtained for $n = 1$ and is called the Euler load.

Table 11–1. Values of k_b and k_s for several beam loadings

Loading	Beam ends	Deflection at	k_b	k_s
Uniformly distributed	Both simply supported	Midspan	5/384	1/8
	Both clamped	Midspan	1/384	1/8
Concentrated at midspan	Both simply supported	Midspan	1/48	1/4
	Both clamped	Midspan	1/192	1/4
Concentrated at outer quarter points	Both simply supported	Midspan	11/76	1/8
	Both simply supported	Load point	1/96	1/8
Uniformly distributed	Cantilever, 1 free, 1 clamped	Free end	1/8	1/2
Concentrated at free end	Cantilever, 1 free, 1 clamped	Free end	1/3	1

At this load, the buckling form is often called "general buckling," as illustrated in Figure 11–12A.

The buckling load N is often expressed in the equivalent form

$$\frac{1}{N} = \frac{1}{N_{\mathrm{E}}} + \frac{1}{U} \qquad (11\text{–}29)$$

When U is finite, $N < N_{\mathrm{E}}$; when U is infinite, $N = N_{\mathrm{E}}$; and when N_{E} is infinite (that is, $n \to \infty$ in Eq. (11–28)), $N = U$, which is often called the "shear instability" limit. The appearance of this buckling failure resembles a crimp (Fig. 11–12B). Shear instability or crimping failure is always possible for edge-loaded sandwich construction and is a limit for general instability and not a localized failure.

For a sandwich panel under edge load and with edge members, the edge members will carry a load proportional to their transformed area (area multiplied by ratio of edge member modulus of elasticity to facing modulus of elasticity). Edge members will also increase the overall panel buckling load because of restraints at edges. Estimates of the effects of edge members can be obtained from Zahn and Cheng (1964).

Buckling criteria for flat rectangular sandwich panels under edgewise shear, bending, and combined loads and those for sandwich walls of cylinders under torsion, axial compression or bending, and external pressure have all been investigated by various researchers at the Forest Products Laboratory. Details can be found in *Military Handbook 23A* by the U.S. Department of Defense (1968) and some publications listed in the References.

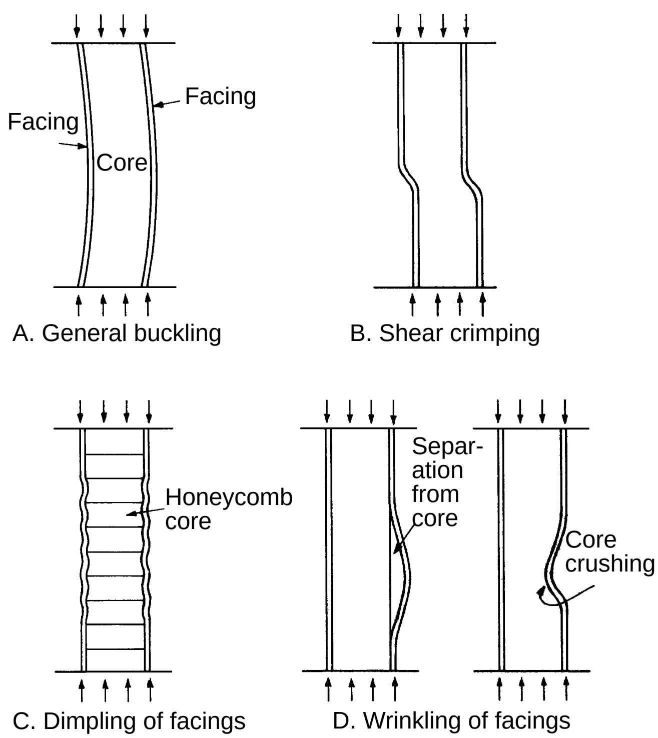

Figure 11–12. Modes of failure of sandwich construction under edgewise loads.

Buckling of sandwich components has been emphasized because it causes complete failure, usually producing severe shear crimping at the edges of the buckles. Another important factor is the necessity that the facing stress be no more than its allowable value at the design load. The facing stress is obtained by dividing the load by the facing area under load. For an edgewise compressive load per unit width N, the facing stress is given by

$$f = \frac{N}{t_1 + t_2} \qquad (11\text{–}30)$$

In a strip of sandwich construction subjected to bending moments, the mean facing stresses are given by

$$f_i = \frac{M}{t_i h} \qquad i = 1,2 \qquad (11\text{–}31)$$

where f_i is mean compressive or tensile stress per unit width in facing i and M is bending moment per unit width of sandwich. If the strip is subjected to shear loads, the shear stress in the core is given by

$$f_{\mathrm{cs}} = \frac{S}{h} \qquad (11\text{–}32)$$

where S is the applied shear load per unit width of sandwich.

Localized failure of sandwich construction must be avoided. Such failure is shown as dimpling and wrinkling of the facings in Figure 11–12C and D, respectively. The stress at which dimpling of the facing into a honeycomb core begins is given by the empirical equation

$$f_{\mathrm{d}} = 2E\left(\frac{t_{\mathrm{f}}}{s}\right)^2 \qquad (11\text{–}33)$$

where f_{d} is facing stress at dimpling, E facing modulus of elasticity at stress f_{d}, t_{f} facing thickness, and s cell size of honeycomb core (diameter of inscribed circle).

Increase in dimpling stress can be attained by decreasing the cell size. Wrinkling of the sandwich facings can occur because they are thin and supported by a lightweight core that functions as their elastic foundation. Wrinkling can also occur because of a poor facing-to-core bond, resulting in separation of facing from the core (Fig. 11–12D). Increase in bond strength should produce wrinkling by core crushing. Thus, a convenient rule of thumb is to require that the sandwich flatwise tensile strength (bond strength) is no less than flatwise compressive core strength. Approximate wrinkling stress for a fairly flat facing (precluding bond failure) is given by

$$f_{\mathrm{w}} = \frac{3}{4}(EE_{\mathrm{c}}G_{\mathrm{c}})^{1/3} \qquad (11\text{–}34)$$

where f_w is facing wrinkling stress, E facing modulus of elasticity, E_c core modulus of elasticity in a direction perpendicular to facing, and G_c core shear modulus.

Wrinkling and other forms of local instability are described in detail in *Military Handbook 23A* (U.S. Department of Defense 1968) and in a book by Allen (1969). Localized failure is not accurately predictable, and designs should be checked by ASTM tests of laboratory specimens.

Because sandwich constructions are composed of several materials, it is often of interest to attempt to design a construction of minimum weight for a particular component. One introduction to the problem of optimum design is presented by Kuenzi (1970). For a sandwich with similar facings having a required bending stiffness D, the dimensions for the minimum weight design are given by

$$h = 2\left(\frac{Dw}{Ew_c}\right)^{1/3} \qquad (11\text{–}35)$$

$$t = \frac{w_c}{4w} h$$

where h is distance between facing centroids, t facing thickness, E facing modulus of elasticity, w facing density, and w_c core density.

The resulting construction will have very thin facings on a very thick core and will be proportioned so that the total core weight is two-thirds the total sandwich weight minus the bond weight. However, such a construction may be impracticable because the required facings may be too thin.

Many detailed design procedures necessary for rapid design of sandwich components for aircraft are summarized in *Military Handbook 23A* (U.S. Department of Defense 1968). The principles contained therein and in some publications listed in the References are broad and can be applied to sandwich components of all structures.

Dimensional Stability, Durability, and Bowing

In a sandwich panel, any dimensional movement of one facing with respect to the other as a result of changes in moisture content and temperature causes bowing of an unrestrained panel. Thus, although the use of dissimilar facings is often desirable from an economic or decorative standpoint, the dimensional instability of the facings during panel manufacture or exposure may rule out possible benefits. If dimensional change of both facings is equal, the length and width of the panel will increase or decrease but bowing will not result.

The problem of dimensional stability is chiefly related to the facings because the core is not stiff enough either to cause bowing of the panel or to cause the panel to remain flat. However, the magnitude of the bowing effect depends on the thickness of the core.

It is possible to calculate mathematically the bowing of a sandwich construction if the percentage of expansion of each facing is known. The maximum deflection is given approximately by

$$\Delta = \frac{ka^2}{800h}$$

where k is the percentage of expansion of one facing compared with the opposite facing, a the length of the panel, and h the distance between facing centroids.

In conventional construction, vapor barriers are often installed to block migration of vapor to the cold side of a wall. Various methods have been tried or suggested for reducing vapor movement through sandwich panels, which causes a moisture differential with resultant bowing of the panels. These methods include bonding metal foil within the sandwich construction, blending aluminum flakes with the resin bonding adhesives, and using plastic vapor barriers between veneers, overlay papers, special finishes, or metal or plastic facings. Because added cost is likely, some methods should not be used unless their need has been demonstrated.

A large test unit simulating the use of sandwich panels in houses was constructed at the Forest Products Laboratory. The panels consisted of a variety of facing materials, including plywood, aluminum, particleboard, hardboard, paperboard, and cement asbestos, with cores of paper honeycomb, polyurethane, or extruded polystyrene. These panels were evaluated for bowing and general performance after various lengths of service between 1947 and 1978. The experimental assembly shown in Figure 11–13 represents the type of construction used in the test unit. The major conclusions were that (a) bowing was least for aluminum-faced panels, (b) bowing was greater for plywood-faced panels with polyurethane or polystyrene cores than for plywood-faced panels with paper cores, and (c) with proper combinations of facings, core, and adhesives, satisfactory sandwich panels can be ensured by careful fabrication techniques.

Thermal Insulation

Satisfactory thermal insulation can best be obtained with sandwich panels by using cores having low thermal conductivity, although the use of reflective layers on the facings is of some value. Paper honeycomb cores have thermal conductivity values (k values), ranging from 0.04 to 0.09 W/m·K (0.30 to 0.65 Btu·in/h·ft^2·°F), depending on the particular core construction. The k value does not vary linearly with core thickness for a true honeycomb core because of direct radiation through the core cell opening from one facing to the other. Honeycomb with open cells can also have greater conductivity if the cells are large enough (greater than about 9 mm (3/8 in.)) to allow convection currents to develop.

An improvement in the insulation value can be realized by filling the honeycomb core with insulation or a foamed-in-place resin.

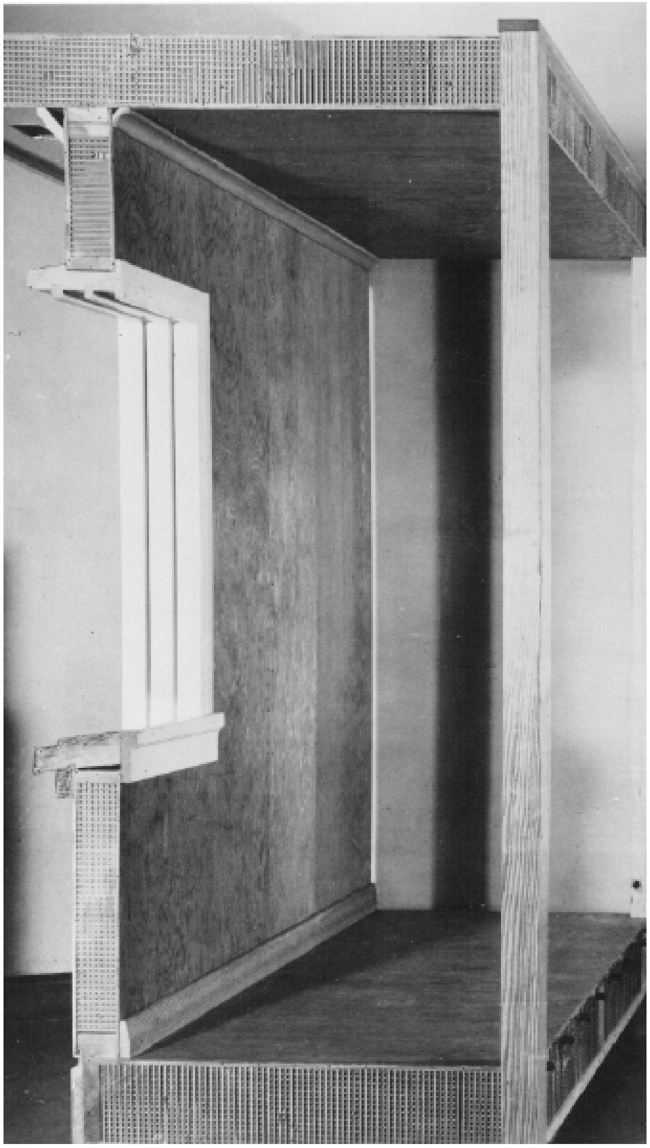

Figure 11–13. Cutaway to show details of sandwich construction in an experimental structure.

Fire Resistance

In tests at the Forest Products Laboratory, the fire resistance of wood-faced sandwich panels was appreciably greater than that of hollow panels faced with the same thickness of plywood. Fire resistance was greatly increased when coatings that intumesce on exposure to heat were applied to the core material. The spread of fire through the honeycomb core depended to a large extent on the alignment of the flutes in the core. In panels with flutes perpendicular to the facings, only slight spread of flame occurred. In cores in which flutes were parallel to the length of the panel, the spread of flame occurred in the vertical direction along open channels. Resistance to flame spread could be improved by placing a barrier sheet at the top of the panel or at intervals in the panel height, or if strength requirements permit, by simply turning the length of the core blocks at 90° angles in the vertical direction.

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Chapter 12

Drying and Control of Moisture Content and Dimensional Changes

William T. Simpson

Contents

In the living tree, wood contains large quantities of water. As green wood dries, most of the water is removed. The moisture remaining in the wood tends to come to equilibrium with the relative humidity of the surrounding air. Correct drying, handling, and storage of wood will minimize moisture content changes that might occur after drying when the wood is in service and such changes are undesirable. If moisture content is controlled within reasonable limits by such methods, major problems from dimensional changes can usually be avoided.

The discussion in this chapter is concerned with moisture content determination, recommended moisture content values, drying methods, methods of calculating dimensional changes, design factors affecting such changes in structures, and moisture content control during transit, storage, and construction. Data on green moisture content, fiber saturation point, shrinkage, and equilibrium moisture content are given with information on other physical properties in Chapter 3.

Wood in service is virtually always undergoing at least slight changes in moisture content. Changes in response to daily humidity changes are small and usually of no consequence. Changes that occur as a result of seasonal variation, although gradual, tend to be of more concern. Protective coatings can retard dimensional changes in wood but do not prevent them. In general, no significant dimensional changes will occur if wood is fabricated or installed at a moisture content corresponding to the average atmospheric conditions to which it will be exposed. When incompletely dried material is used in construction, some minor dimensional changes can be tolerated if the proper design is used.

Determination of Moisture Content

The amount of moisture in wood is ordinarily expressed as a percentage of the weight of the wood when ovendry. Four methods of determining moisture content are covered in ASTM D4442. Two of these—the oven-drying and the electrical methods—are described in this chapter.

The oven-drying method has been the most universally accepted method for determining moisture content, but it is slow and necessitates cutting the wood. In addition, the oven-drying method may give values slightly greater than true moisture content with woods containing volatile extractives. The electrical method is rapid, does not require cutting the wood, and can be used on wood in place in a structure. However, considerable care must be taken to use and interpret the results correctly. Use of the electrical method is generally limited to moisture content values less than 30%.

Oven-Drying Method

In the oven-drying method, specimens are taken from representative boards or pieces of a quantity of lumber. With lumber, the specimens should be obtained at least 500 mm (20 in.) from the end of the pieces. They should be free from knots and other irregularities, such as bark and pitch pockets. Specimens from lumber should be full cross sections and 25 mm (1 in.) long. Specimens from larger items may be representative sectors of such sections or subdivided increment borer or auger chip samples. Convenient amounts of chips and particles can be selected at random from larger batches, with care taken to ensure that the sample is representative of the batch. Veneer samples should be selected from four or five locations in a sheet to ensure that the sample average will accurately indicate the average of the sheet.

Each specimen should be weighed immediately, before any drying or reabsorption of moisture has taken place. If the specimen cannot be weighed immediately, it should be placed in a plastic bag or tightly wrapped in metal foil to protect it from moisture change until it can be weighed. After weighing, the specimen is placed in an oven heated to 101°C to 105°C (214°F to 221°F) and kept there until no appreciable weight change occurs in 4-h weighing intervals. A lumber section 25 mm (1 in.) along the grain will reach a constant weight in 12 to 48 h. Smaller specimens will take less time. The constant or ovendry weight and the weight of the specimen when cut are used to determine the percentage of moisture content using the formula

$$\text{Moisture content (\%)} = \frac{\text{Weight when cut} - \text{Ovendry weight}}{\text{Ovendry weight}} \times 100 \qquad (12\text{–}1)$$

Electrical Method

The electrical method of determining the moisture content of wood uses the relationships between moisture content and measurable electrical properties of wood, such as conductivity (or its inverse, resistivity), dielectric constant, or power-loss factor. These properties vary in a definite and predictable way with changing moisture content, but correlations are not perfect. Therefore, moisture determinations using electrical methods are always subject to some uncertainty.

Electric moisture meters are available commercially and are based on each of these properties and identified by the property measured. Conductance-type (or resistance) meters measure moisture content in terms of the direct current conductance of the specimen. Dielectric-type meters are of two types. Those based principally on dielectric constant are called capacitance or capacitive admittance meters; those based on loss factor are called power-loss meters.

The principal advantages of the electrical method compared with the oven-drying method are speed and convenience. Only a few seconds are required for the determination, and the piece of wood being tested is not cut or damaged, except for driving electrode needle points into the wood when using conductance-type meters. Thus, the electrical method is adaptable to rapid sorting of lumber on the basis of moisture content, measuring the moisture content of wood installed in a building, or establishing the moisture content of a quantity of lumber or other wood items, when used in accordance with ASTM D4442.

For conductance meters, needle electrodes of various lengths are driven into the wood. There are two general types of electrodes: insulated and uninsulated. Uninsulated electrodes will sense the highest moisture content along their length (highest conductance). Moisture gradients between the surface and the interior can lead to confusion. If the wood is wetter near the center than the surface, which is typical for drying wood, the reading will correspond to the depth of the tip of the insulated electrodes. If a meter reading increases as the electrodes are being driven in, then the moisture gradient is typical. In this case, the pins should be driven in about one-fifth to one-fourth the thickness of the wood to reflect the average moisture content of the entire piece. Dried or partially dried wood sometimes regains moisture in the surface fibers, and the surface moisture content is greater than the interior. In this case, the meter with the uninsulated pins will read the higher moisture content surface, possibly causing a significant deviation from the average moisture content. To guard against this problem, electrodes with insulated shanks have been developed. They measure moisture content of only the wood at the tips of the electrodes.

Dielectric-type meters are fitted with surface contact electrodes designed for the type of specimen material being tested. The electric field from these electrodes penetrates well into the specimen, but with a strength that decreases rapidly with depth of penetration. For this reason, the readings of dielectric meters are influenced predominantly by the surface layers of the specimen, and the material near midthickness may not be adequately represented in the meter reading if there is a moisture content gradient.

To obtain accurate moisture content values, each instrument should be used in accordance with its manufacturer's instructions. The electrodes should be appropriate for the material being tested and properly oriented according to meter manufacturer's instructions. The readings should be carefully taken as soon as possible after inserting the electrode. A species correction supplied with the instrument should be applied when appropriate. Temperature corrections should then be made if the temperature of the wood differs considerably from the temperature of calibration used by the manufacturer.

Approximate corrections for conductance-type (resistance) meters are made by adding or subtracting about 0.5% for each 5.6°C (10°F) the wood temperature differs from the calibration temperature. The correction factors are added to the readings for temperatures less than the calibration temperature and subtracted from the readings for temperatures greater than the calibration temperature. Temperature corrections for dielectric meters are rather complex and are best made from published charts (James 1988).

Although some meters have scales that go up to 120%, the range of moisture content that can be measured reliably is 4% to about 30% for commercial dielectric meters and about 6% to 30% for resistance meters. The precision of the individual meter readings decreases near the limits of these ranges. Readings greater than 30% must be considered only qualitative. When the meter is properly used on a quantity of lumber dried to a reasonably constant moisture content below fiber saturation, the average moisture content from the corrected meter readings should be within 1% of the true average.

Recommended Moisture Content

Wood should be installed at moisture content levels as close as possible to the average moisture content it will experience in service. This minimizes the seasonal variation in moisture content and dimension after installation, avoiding problems such as floor buckling or cracks in furniture. The in-service moisture content of exterior wood (siding, wood trim) primarily depends on the outdoor relative humidity and exposure to rain or sun. The in-service moisture content of interior wood primarily depends on indoor relative humidity, which in turn is a complex function of moisture sources, ventilation rate, dehumidification (for example, air conditioning), and outdoor humidity conditions.

The recommended values for interior wood presented in this chapter are based on measurements in well-ventilated buildings without unusual moisture sources and without air conditioning. In air-conditioned buildings, moisture conditions depend to a great extent on the proper sizing of the air-conditioning equipment. Wood installed in basements or over a crawl space may experience a moisture content greater than the range provided, and wood in insulated walls or roofs and attics may experience a moisture content greater or less than the range. Nevertheless, the recommended values for installation provide a useful guideline.

Timbers

Ideally, solid timbers should be dried to the average moisture content they will reach in service. Although this optimum is possible with lumber less than 76 mm (3 in.) thick, it is seldom practical to obtain fully dried timbers, thick joists, and planks. When thick solid members are used, some shrinkage of the assembly should be expected. In the case of built-up assemblies, such as roof trusses, it may be necessary to tighten bolts or other fastenings occasionally to maintain full bearing of the connectors as the members shrink.

Lumber

The recommended moisture content of wood should be matched as closely as is practical to the equilibrium moisture content (EMC) conditions in service. Table 12–1 shows the EMC conditions in outdoor exposure in various U.S. cities for each month. The EMC data are based on the average relative humidity and temperature data (30 or more years) available from the National Climatic Data Center of the National Oceanic and Atmospheric Administration. The relative humidity data were the average of the morning and afternoon values, and in most cases would be representative of the EMC attained by the wood. However, in some locations, early morning relative humidity may occasionally reach 100%. Under these conditions, condensation may occur and the surface fibers of wood will exceed the EMC. The moisture content requirements are more exacting for finished lumber and wood products used inside heated and air-conditioned buildings than those for lumber used outdoors or in unheated buildings. For general areas of the United States, the recommended moisture content values for wood used inside heated buildings are shown in Figure 12–1. Values and tolerances for both interior and exterior uses of wood in various forms are given in Table 12–2. If the average moisture content is within 1% of that recommended and all pieces fall within the individual limits, the entire lot is probably satisfactory.

General commercial practice is to kiln dry wood for some products, such as flooring and furniture, to a slightly lower moisture content than service conditions demand, anticipating a moderate increase in moisture content during processing and construction. This practice is intended to ensure uniform distribution of moisture among the individual pieces. Common grades of softwood lumber and softwood dimension lumber are not normally dried to the moisture content values indicated in Table 12–2. Dry lumber, as defined in the American Softwood Lumber Standard, has a maximum moisture content of 19%. Some industry grading rules provide for an even lower maximum. For example, to be grade marked KD 15, the maximum moisture content permitted is generally 15%.

Glued Wood Products

When veneers are bonded with cold-setting adhesives to make plywood, they absorb comparatively large quantities of moisture. To keep the final moisture content low and to minimize redrying of the plywood, the initial moisture content of the veneer should be as low as practical. However, very dry veneer is brittle and difficult to handle without damage, so the minimum practical moisture content is about 4%. Freshly glued plywood intended for interior service should be dried to the moisture content values given in Table 12–2.

Table 12–1. Equilibrium moisture content of wood, exposed to outdoor atmosphere, in several U.S. locations in 1997

		Equilibrium moisture content[a] (%)											
State	City	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
AK	Juneau	16.5	16.0	15.1	13.9	13.6	13.9	15.1	16.5	18.1	18.0	17.7	18.1
AL	Mobile	13.8	13.1	13.3	13.3	13.4	13.3	14.2	14.4	13.9	13.0	13.7	14.0
AZ	Flagstaff	11.8	11.4	10.8	9.3	8.8	7.5	9.7	11.1	10.3	10.1	10.8	11.8
AZ	Phoenix	9.4	8.4	7.9	6.1	5.1	4.6	6.2	6.9	6.9	7.0	8.2	9.5
AR	Little Rock	13.8	13.2	12.8	13.1	13.7	13.1	13.3	13.5	13.9	13.1	13.5	13.9
CA	Fresno	16.4	14.1	12.6	10.6	9.1	8.2	7.8	8.4	9.2	10.3	13.4	16.6
CA	Los Angeles	12.2	13.0	13.8	13.8	14.4	14.8	15.0	15.1	14.5	13.8	12.4	12.1
CO	Denver	10.7	10.5	10.2	9.6	10.2	9.6	9.4	9.6	9.5	9.5	11.0	11.0
DC	Washington	11.8	11.5	11.3	11.1	11.6	11.7	11.7	12.3	12.6	12.5	12.2	12.2
FL	Miami	13.5	13.1	12.8	12.3	12.7	14.0	13.7	14.1	14.5	13.5	13.9	13.4
GA	Atlanta	13.3	12.3	12.0	11.8	12.5	13.0	13.8	14.2	13.9	13.0	12.9	13.2
HI	Honolulu	13.3	12.8	11.9	11.3	10.8	10.6	10.6	10.7	10.8	11.3	12.1	12.9
ID	Boise	15.2	13.5	11.1	10.0	9.7	9.0	7.3	7.3	8.4	10.0	13.3	15.2
IL	Chicago	14.2	13.7	13.4	12.5	12.2	12.4	12.8	13.3	13.3	12.9	14.0	14.9
IN	Indianapolis	15.1	14.6	13.8	12.8	13.0	12.8	13.9	14.5	14.2	13.7	14.8	15.7
IA	Des Moines	14.0	13.9	13.3	12.6	12.4	12.6	13.1	13.4	13.7	12.7	13.9	14.9
KS	Wichita	13.8	13.4	12.4	12.4	13.2	12.5	11.5	11.8	12.6	12.4	13.2	13.9
KY	Louisville	13.7	13.3	12.6	12.0	12.8	13.0	13.3	13.7	14.1	13.3	13.5	13.9
LA	New Orleans	14.9	14.3	14.0	14.2	14.1	14.6	15.2	15.3	14.8	14.0	14.2	15.0
ME	Portland	13.1	12.7	12.7	12.1	12.6	13.0	13.0	13.4	13.9	13.8	14.0	13.5
MA	Boston	11.8	11.6	11.9	11.7	12.2	12.1	11.9	12.5	13.1	12.8	12.6	12.2
MI	Detroit	14.7	14.1	13.5	12.6	12.3	12.3	12.6	13.3	13.7	13.5	14.4	15.1
MN	Minneapolis–St.Paul	13.7	13.6	13.3	12.0	11.9	12.3	12.5	13.2	13.8	13.3	14.3	14.6
MS	Jackson	15.1	14.4	13.7	13.8	14.1	13.9	14.6	14.6	14.6	14.1	14.3	14.9
MO	St. Louis	14.5	14.1	13.2	12.4	12.8	12.6	12.9	13.3	13.7	13.1	14.0	14.9
MT	Missoula	16.7	15.1	12.8	11.4	11.6	11.7	10.1	9.8	11.3	12.9	16.2	17.6
NE	Omaha	14.0	13.8	13.0	12.1	12.6	12.9	13.3	13.8	14.0	13.0	13.9	14.8
NV	Las Vegas	8.5	7.7	7.0	5.5	5.0	4.0	4.5	5.2	5.3	5.9	7.2	8.4
NV	Reno	12.3	10.7	9.7	8.8	8.8	8.2	7.7	7.9	8.4	9.4	10.9	12.3
NM	Albuquerque	10.4	9.3	8.0	6.9	6.8	6.4	8.0	8.9	8.7	8.6	9.6	10.7
NY	New York	12.2	11.9	11.5	11.0	11.5	11.8	11.8	12.4	12.6	12.3	12.5	12.3
NC	Raleigh	12.8	12.1	12.2	11.7	13.1	13.4	13.8	14.5	14.5	13.7	12.9	12.8
ND	Fargo	14.2	14.6	15.2	12.9	11.9	12.9	13.2	13.2	13.7	13.5	15.2	15.2
OH	Cleveland	14.6	14.2	13.7	12.6	12.7	12.7	12.8	13.7	13.8	13.3	13.8	14.6
OK	Oklahoma City	13.2	12.9	12.2	12.1	13.4	13.1	11.7	11.8	12.9	12.3	12.8	13.2
OR	Pendleton	15.8	14.0	11.6	10.6	9.9	9.1	7.4	7.7	8.8	11.0	14.6	16.5
OR	Portland	16.5	15.3	14.2	13.5	13.1	12.4	11.7	11.9	12.6	15.0	16.8	17.4
PA	Philadelphia	12.6	11.9	11.7	11.2	11.8	11.9	12.1	12.4	13.0	13.0	12.7	12.7
SC	Charleston	13.3	12.6	12.5	12.4	12.8	13.5	14.1	14.6	14.5	13.7	13.2	13.2
SD	Sioux Falls	14.2	14.6	14.2	12.9	12.6	12.8	12.6	13.3	13.6	13.0	14.6	15.3
TN	Memphis	13.8	13.1	12.4	12.2	12.7	12.8	13.0	13.1	13.2	12.5	12.9	13.6
TX	Dallas–Ft.Worth	13.6	13.1	12.9	13.2	13.9	13.0	11.6	11.7	12.9	12.8	13.1	13.5
TX	El Paso	9.6	8.2	7.0	5.8	6.1	6.3	8.3	9.1	9.3	8.8	9.0	9.8
UT	Salt Lake City	14.6	13.2	11.1	10.0	9.4	8.2	7.1	7.4	8.5	10.3	12.8	14.9
VA	Richmond	13.2	12.5	12.0	11.3	12.1	12.4	13.0	13.7	13.8	13.5	12.8	13.0
WA	Seattle–Tacoma	15.6	14.6	15.4	13.7	13.0	12.7	12.2	12.5	13.5	15.3	16.3	16.5
WI	Madison	14.5	14.3	14.1	12.8	12.5	12.8	13.4	14.4	14.9	14.1	15.2	15.7
WV	Charleston	13.7	13.0	12.1	11.4	12.5	13.3	14.1	14.3	14.0	13.6	13.0	13.5
WY	Cheyenne	10.2	10.4	10.7	10.4	10.8	10.5	9.9	9.9	9.7	9.7	10.6	10.6

[a]EMC values were determined from the average of 30 or more years of relative humidity and temperature data available from the National Climatic Data Center of the National Oceanic and Atmospheric Administration.

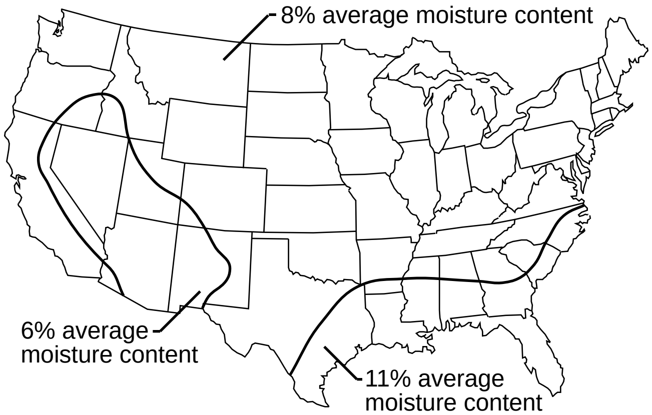

Figure 12–1. Recommended average moisture content for interior use of wood products in various areas of the United States.

Hot-pressed plywood and other board products, such as particleboard and hardboard, usually do not have the same moisture content as lumber. The high temperatures used in hot presses cause these products to assume a lower moisture content for a given relative humidity. Because this lower equilibrium moisture content varies widely, depending on the specific type of hot-pressed product, it is recommended that such products be conditioned at 30% to 40% relative humidity for interior use and 65% for exterior use.

Lumber used in the manufacture of large laminated members should be dried to a moisture content slightly less than the moisture content expected in service so that moisture absorbed from the adhesive will not cause the moisture content of the product to exceed the service value. The range of moisture content between laminations assembled into a single member should not exceed 5 percentage points. Although laminated members are often massive and respond rather slowly to changes in environmental conditions, it is desirable to follow the recommendations in Table 12–2 for moisture content at time of installation.

Drying of Wood

Drying is required for wood to be used in most products. Dried lumber has many advantages over green lumber for producers and consumers. Removal of excess water reduces weight, thus shipping and handling costs. Proper drying confines shrinking and swelling of wood in use to manageable amounts under all but extreme conditions of relative humidity or flooding. As wood dries, most of its strength properties increase, as well as its electrical and thermal insulating properties. Properly dried lumber can be cut to precise dimensions and machined more easily and efficiently; wood parts can be more securely fitted and fastened together with nails, screws, bolts, and adhesives; warping, splitting, checking, and other harmful effects of uncontrolled drying are largely eliminated; and paint, varnish, and other finishes are more effectively applied and maintained. Wood must be relatively dry before it can be glued or treated with decay-preventing and fire-retardant chemicals.

The key to successful and efficient drying is control of the drying process. Timely application of optimum or at least adequate temperature, relative humidity, and air circulation conditions is critical. Uncontrolled drying leads to drying defects that can adversely affect the serviceability and economics of the product. The usual strategy is to dry as fast as the particular species, thickness, and end-product requirements allow without damaging the wood. Slower drying can be uneconomical as well as introduce the risk of stain.

Softwood lumber intended for framing in construction is usually targeted for drying to an average moisture content of 15%, not to exceed 19%. Softwood lumber for many other uses is dried to a low moisture content, 10% to 12% for many appearance grades to as low as 7% to 9% for furniture, cabinets, and millwork. Hardwood lumber for framing in construction, although not in common use, should also be dried to an average moisture content of 15%, not to exceed 19%. Hardwood lumber for furniture, cabinets, and millwork is usually dried to 6% to 8% moisture content.

Table 12–2. Recommended moisture content values for various wood items at time of installation

	Recommended moisture content (%) in various climatological regions					
	Most areas of the United States		Dry southwestern area[a]		Damp, warm coastal area[a]	
Use of wood	Average[b]	Individual pieces	Average[b]	Individual pieces	Average[b]	Individual pieces
Interior: woodwork, flooring, furniture, wood trim	8	6–10	6	4–9	11	8–13
Exterior: siding, wood trim, sheathing, laminated timbers	12	9–14	9	7–12	12	9–14

[a]Major areas are indicated in Figure 12–1.
[b]To obtain a realistic average, test at least 10% of each item. If the quantity of a given item is small, make several tests. For example, in an ordinary dwelling having about 60 floor joists, at least 10 tests should be made on joists selected at random.

Lumber drying is usually accomplished by some combination of air drying, accelerated air drying or pre-drying, and kiln drying. Wood species, lumber thickness, economics, and end use are often the main factors in determining the details of the drying process.

Air Drying

The main purpose of air drying lumber is to evaporate as much of the water as possible before end use or transfer to a dry kiln. Air drying usually extends until wood moisture content is as low as 20% to 25%, at which time the lumber is transferred to a dry kiln if final drying to a lower moisture content is required. Sometimes, depending on a mill's scheduling, air drying may be cut short at a higher moisture content before the wood is sent to the dry kiln. Air drying saves energy costs and reduces required dry kiln capacity. Limitations of air drying are generally associated with uncontrolled drying. The drying rate is very slow during the cold winter months. At other times, hot, dry winds may increase degrade and volume losses as a result of severe surface checking and end splitting. Warm, humid periods with little air movement may encourage the growth of fungal stains, as well as aggravate chemical stains. Another limitation of air drying is the high cost of carrying a large inventory of high value lumber for extended periods. Air drying time to 20% to 25% moisture content varies widely, depending on species, thickness, location, and the time of year the lumber is stacked. Some examples of extremes for 25-mm- (1-in.-) thick lumber are 15 to 30 days for some of the low density species, such as pine, spruce, red alder, and soft maple, stacked in favorable locations and favorable times of the year, to 200 to 300 days for slow drying species, such as sinker hemlock and pine, oak, and birch, in northern locations and stacked at unfavorable times of the year. Details of important air drying considerations, such as lumber stacking and air drying yard layout, are covered in *Air Drying of Lumber: A Guide to Industry Practices* (Rietz and Page 1971).

Accelerated Air Drying and Pre-Drying

The limitations of air drying have led to increased use of technology that reduces drying time and introduces some control into drying from green to 20% to 25% moisture content. Accelerated air drying involves the use of fans to force air through lumber piles in a shed. This protects the lumber from the elements and improves air circulation compared with air drying. Small amounts of heat are sometimes used to reduce relative humidity and slightly increase temperature. Pre-dryers take this acceleration and control a step further by providing control of both temperature and relative humidity and providing forced air circulation in a completely enclosed compartment. Typical conditions in a pre-dryer are 27°C to 38°C (80°F to 100°F) and 65% to 85% relative humidity.

Kiln Drying

In kiln drying, higher temperatures and faster air circulation are used to increase drying rate considerably. Specific kiln schedules have been developed to control temperature and relative humidity in accordance with the moisture content and stress situation within the wood, thus minimizing shrinkage-caused defects.

Drying Mechanism

Water in wood normally moves from high to low zones of moisture content, which means that the surface of the wood must be drier than the interior if moisture is to be removed. Drying can be broken down into two phases: movement of water from the interior to the surface of the wood and evaporation of water from the surface. The surface fibers of most species reach moisture equilibrium with the surrounding air soon after drying begins. This is the beginning of the development of a typical moisture gradient (Fig. 12–2), that is, the difference in moisture content between the inner and outer portions of a board. If air circulation is too slow, a longer time is required for the surfaces of the wood to reach moisture equilibrium. This is one reason why air circulation is so important in kiln drying. If air circulation is too slow, drying is also slower than necessary and mold could develop on the surface of lumber. If drying is too fast, electrical energy in running the fans is wasted, and in certain species,

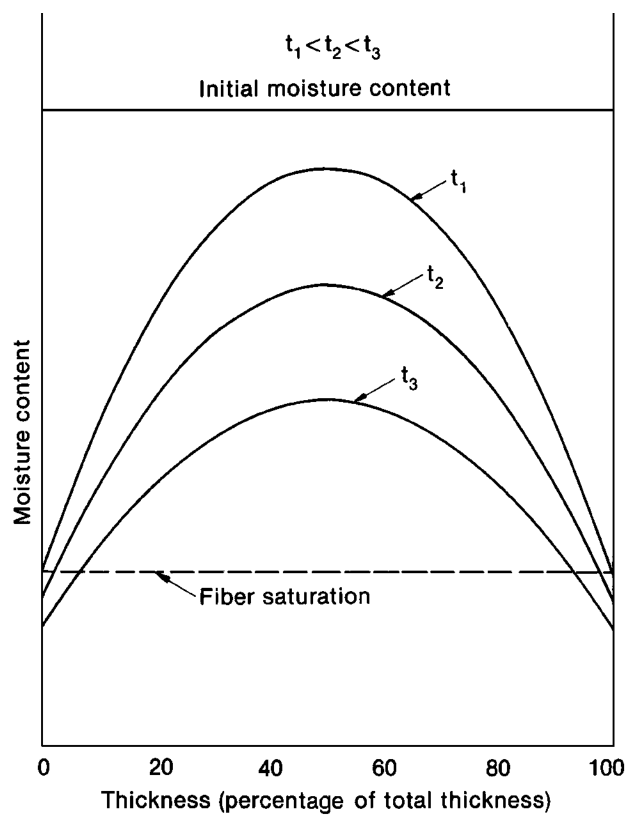

Figure 12–2. Typical moisture gradient in lumber during drying at time increasing from t_1 to t_3.

surface checking and other drying defects can develop if relative humidity and air velocity are not coordinated.

Water moves through the interior of wood as a liquid or vapor through various air passageways in the cellular structure of the wood, as well as through the wood cell walls. Moisture moves in these passageways in all directions, both across and with the grain. In general, lighter species dry faster than heavier species because the structure of lighter wood contains more openings per unit volume and moisture moves through air faster than through wood cell walls. Water moves by two main mechanisms: capillary action (liquid) and diffusion of bound water (vapor). Capillary action causes free water to flow through cell cavities and the small passageways that connect adjacent cell cavities. Diffusion of bound water moves moisture from areas of high concentration to areas of low concentration. Diffusion in the longitudinal direction is about 10 to 15 times faster than radial or tangential diffusion, and radial diffusion is somewhat faster than tangential diffusion. This explains why flatsawn lumber generally dries faster than quartersawn lumber. Although longitudinal diffusion is much faster than diffusion across the grain, it generally is not of practical importance in lumber that is many times longer than it is thick.

Because chemical extractives in heartwood plug up passageways, moisture generally moves more freely in sapwood than in heartwood; thus, sapwood generally dries faster than heartwood. However, the heartwood of many species is lower in moisture content than is the sapwood and can reach final moisture content as fast.

The rate at which moisture moves in wood depends on the relative humidity of the surrounding air, the steepness of the moisture gradient, and the temperature of the wood. The lower the relative humidity, the greater the capillary flow. Low relative humidity also stimulates diffusion by lowering the moisture content at the surface, thereby steepening the moisture gradient and increasing the diffusion rate. The greater the temperature of the wood, the faster moisture will move from the wetter interior to the drier surface. If relative humidity is too low in the early stages of drying, excessive shrinkage may occur, resulting in surface and end checking. If the temperature is too high, collapse, honeycomb, or strength reduction can occur.

Drying Stresses

Drying stresses are the main cause of nonstain-related drying defects. Understanding these stresses provides a means for minimizing and recognizing the damage they can cause. The cause of drying stresses in the differential shrinkage between the outer part of a board (the shell) and the interior part (the core) can also cause drying defects. Early in drying, the fibers in the shell dry first and begin to shrink. However, the core has not yet begun to dry and shrink; consequently, the core prevents the shell from shrinking. Thus, the shell goes into tension and the core into compression (Fig. 12–3). If the shell dries too rapidly, it is stressed beyond the elastic limit and dries in a permanently stretched (set) condition without attaining full shrinkage. Sometimes surface cracks,

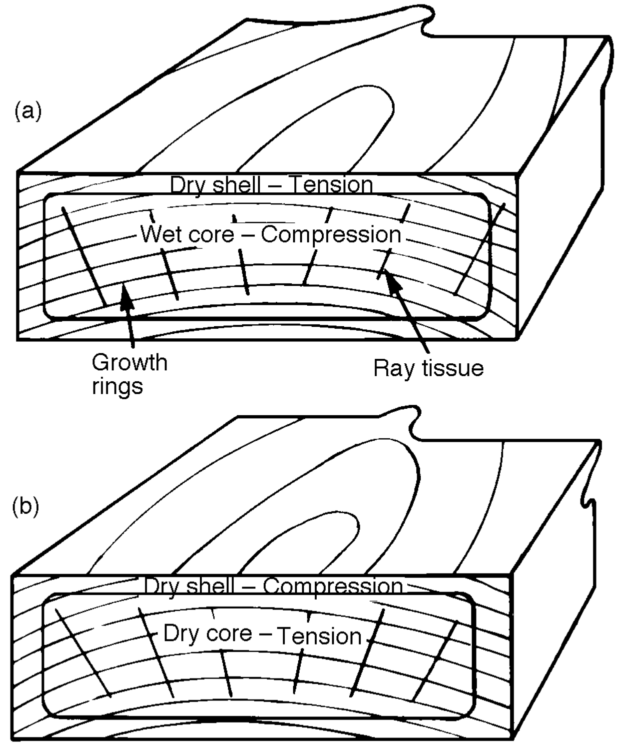

Figure 12–3. End view of board showing development of drying stresses (a) early and (b) later in drying.

or checks, occur during this initial stage of drying, and they can be a serious defect for many uses. As drying progresses, the core begins to dry and attempts to shrink. However, the shell is set in a permanently expanded condition and prevents normal shrinkage of the core. This causes the stresses to reverse; the core goes into tension and the shell into compression. The change in the shell and core stresses and in the moisture content level during drying is shown in Figure 12–4. These internal tension stresses may be severe enough to cause internal cracks (honeycomb).

Differential shrinkage caused by differences in radial, tangential, and longitudinal shrinkage is a major cause of warp. The distortions shown in Figure 3–3 in Chapter 3 are due to differential shrinkage. When juvenile or reaction wood is present on one edge or face of a board and normal wood is present on the opposite side, the difference in their longitudinal shrinkage can also cause warp.

Dry Kilns

Most dry kilns are thermally insulated compartments designed for a batch process in which the kiln is completely loaded with lumber in one operation and the lumber remains stationary during the entire drying cycle. Temperature and relative humidity are kept as uniform as possible throughout the kiln and can be controlled over a wide range. Temperature and relative humidity are changed as the wood dries based on a schedule that takes into account the moisture content and/or the drying rate of the lumber. All dry kilns

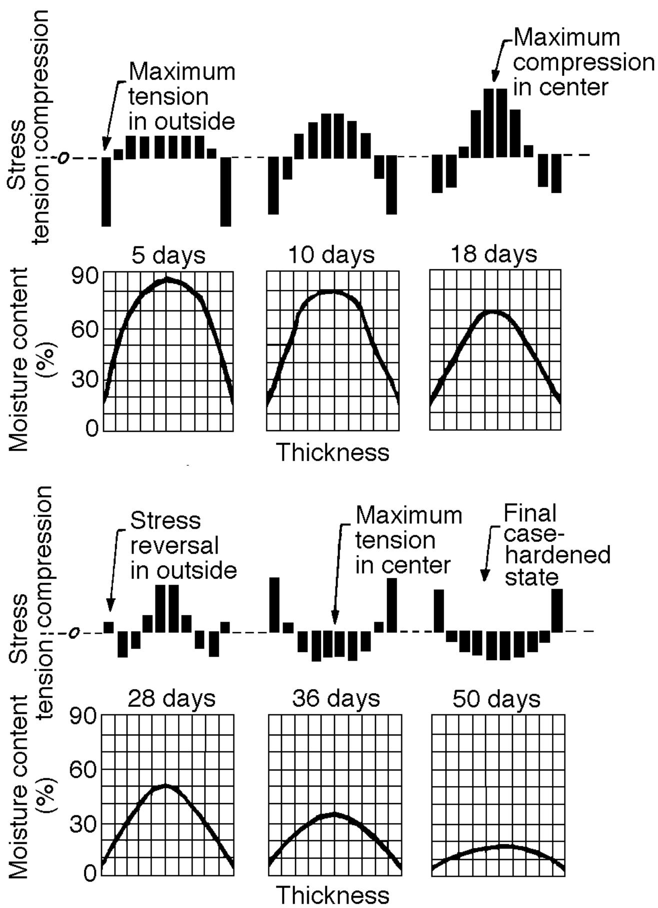

Figure 12–4. Moisture–stress relationship during six stages of kiln drying 50-mm- (2-in.-) thick red oak.

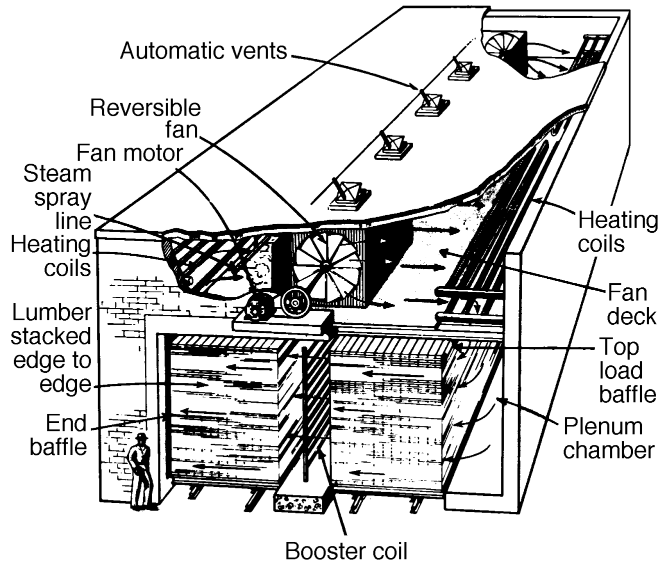

Figure 12–5. Lineshaft, double-track, compartment kiln with alternately opposing fans. Vents are over fan shaft between fans. Vent on high pressure side of fans becomes fresh air inlet when direction of circulation is reversed.

use some type of forced-air circulation, with air moving through the lumber perpendicular to the length of the lumber and parallel to the spacers (stickers) that separate each layer of lumber in a stack.

Three general types of kilns are in common use. One is the track-loaded type (Fig. 12–5), where lumber is stacked on kiln trucks that are rolled in and out of the kiln on tracks. The majority of softwood lumber in the United States is dried in this kiln type. Another major type is the package-loaded kiln (Fig 12–6), where individual stacks of lumber are fork-lifted into place in the kiln. This type of kiln is commonly used for drying hardwood lumber. These kilns are most commonly heated with steam, although softwood lumber kilns are sometimes directly heated. A third common type of kiln, usually package loaded, is the dehumidification kiln. Instead of venting humid air to remove water, as the other two types of kilns do, water is removed by condensation on cold dehumidifier coils (Fig. 12–7).

Kiln Schedules

A kiln schedule is a carefully developed compromise between the need to dry lumber as fast as possible for economic efficiency and the need to avoid severe drying conditions that will lead to drying defects. A kiln schedule is a series of temperatures and relative humidities that are applied at various stages of drying. In most schedules, the temperature is gradually increased and the relative humidity decreased. The schedule for Southern Pine structural lumber is an exception to this general rule. This is lumber usually dried at a constant temperature and relative humidity. Temperatures are chosen to strike this compromise of a satisfactory drying rate and avoidance of objectionable drying defects. The stresses that develop during drying are the limiting factor in determining the kiln schedule. The schedule must be developed so that the drying stresses do not exceed the strength of the wood at any given temperature and moisture content. Otherwise, the wood will crack either on the surface or internally or be crushed by forces that collapse the wood cells. Wood generally becomes stronger as the moisture content decreases, and to a lesser extent, it becomes weaker as temperature increases. The net result is that as wood dries it becomes stronger because of the decreasing moisture content and can tolerate higher drying temperatures and lower relative humidities without cracking. This is a fortunate circumstance because as wood dries, its drying rate decreases at any given temperature, and the ability to increase drying temperature helps maintain a reasonably fast drying rate. Thus, rapid drying is achieved in kilns by the use of temperatures as high as possible and relative humidities as low as possible.

Drying schedules vary by species, thickness, grade, and end use of lumber. There are two general types of kiln schedules: moisture content schedules and time-based schedules. Most hardwood lumber is dried by moisture content schedules. This means that the temperature and relative humidity conditions are changed according to various moisture content

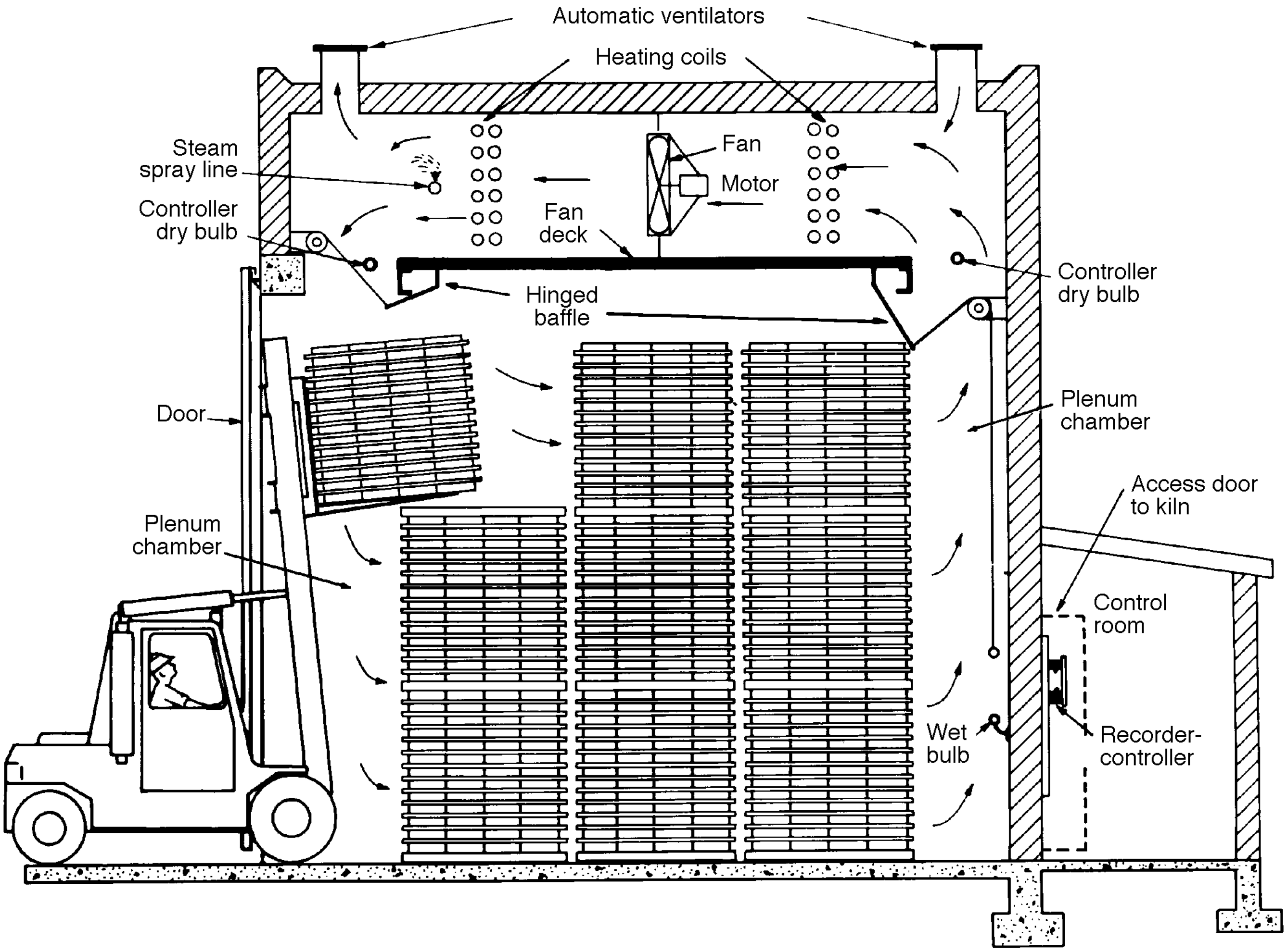

Figure 12–6. Package-loaded kiln with fans connected directly to motors.

levels attained by the lumber during drying. A typical hardwood schedule might begin at 49°C (120°F) and 80% relative humidity when the lumber is green. By the time the lumber has reached 15% moisture content, the temperature is as high as 82°C (180°F). A typical hardwood drying schedule is shown in Table 12–3. Some method of monitoring moisture content during drying is required for schedules based on moisture content. One common method is the use of short kiln samples that are periodically weighed, usually manually but potentially remotely with load cells. Alternatively, electrodes are imbedded in sample boards to sense the change in electrical conductivity with moisture content. This system is limited to moisture content values less than 30%.

Softwood kiln schedules generally differ from hardwood schedules in that changes in kiln temperature and relative humidity are made at predetermined times rather than moisture content levels. Examples of time-based schedules, both conventional temperature (<100°C (<212°F)) and high temperature (>110°C (>230°F)), are given in Table 12–3.

Drying Defects

Most drying defects or problems that develop in wood products during drying can be classified as fracture or distortion, warp, or discoloration. Defects in any one of these categories are caused by an interaction of wood properties with processing factors. Wood shrinkage is mainly responsible for wood ruptures and distortion of shape. Cell structure and chemical extractives in wood contribute to defects associated with uneven moisture content, undesirable color, and undesirable surface texture. Drying temperature is the most important processing factor because it can be responsible for defects in each category.

Fracture or Distortion

Surface checks occur early in drying when the shell of a board is stressed in tension enough to fracture the wood. These checks occur most often on the face of flatsawn boards and are illustrated in Figure 12–8. End checks (Fig. 12–9) are similar to surface checks but appear on the ends of boards. End checks occur because the rapid longitudinal movement of moisture causes the board end to dry very quickly and

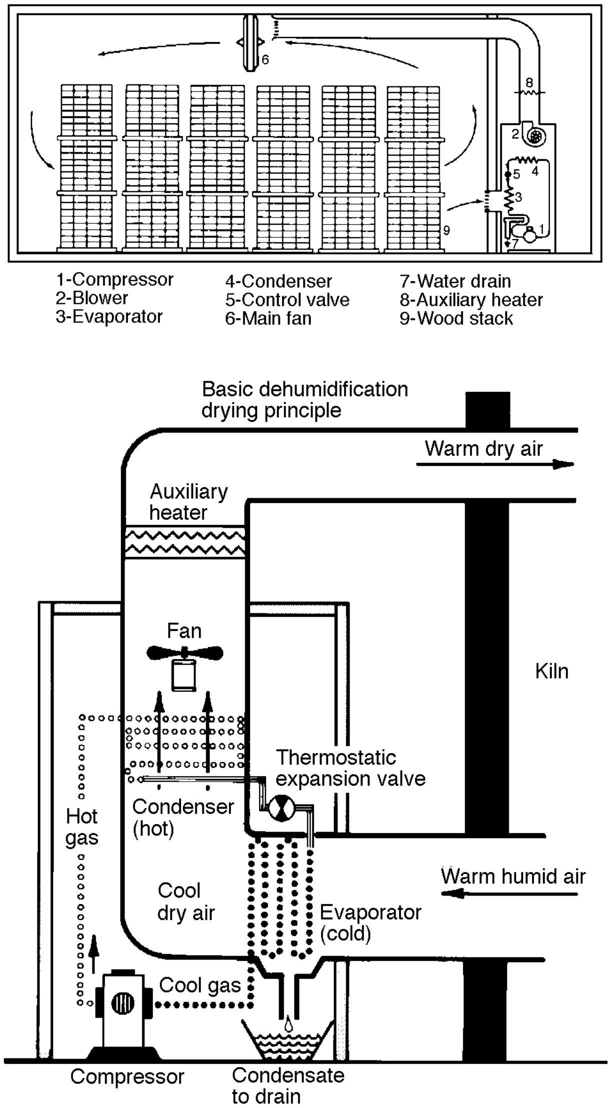

Fig. 12–7. A typical dehumidification kiln (top) and dehumidification drying system (bottom).

develop high stresses, therefore fracturing. End coatings, either on the log or freshly sawn lumber, are an effective preventative measure. Collapse is a distortion, flattening, or crushing of wood cells. In severe cases (Fig. 12–10), collapse usually shows up as grooves or corrugations, a washboarding effect. Less severe collapse shows up as excessive thickness shrinkage and may not be a serious problem. Honeycomb (Fig. 12–11) is an internal crack that occurs in the later stages of kiln drying when the core of a board is in tension. It is caused when the core is still at a relatively high moisture content and drying temperatures are too high for too long during this critical drying period. Nondestructive testing methods, using speed of sound, have been found to be effective in detecting the presence of these cracks in dried lumber. Knots may loosen during drying because of the unequal shrinkage between the knot and the surrounding wood (Fig. 12–12).

Warp

Warp in lumber is any deviation of the face or edge of a board from flatness or any edge that is not at right angles to the adjacent face or edge. Warp can be traced to two causes: (a) differences between radial, tangential, and longitudinal shrinkage in the piece as it dries or (b) growth stresses. Warp is aggravated by irregular or distorted grain and the presence of abnormal types of wood, such as juvenile and reaction wood. The six major types of warp are bow, crook, twist, oval, diamond, and cup (Fig. 12–13).

Discoloration

The use of dried wood products can be impaired by discoloration, particularly when the end use requires a clear, natural finish. Unwanted discoloration can develop in the tree, during storage of logs and green lumber, or during drying. There are two general types of discoloration: chemical and fungal.

Chemical discoloration is the result of oxidative and enzymatic reactions with chemical constituents in wood. Discolorations range from pinkish, bluish, and yellowish hues through gray and reddish brown to dark brown shades. Brown stain in pines and darkening in many hardwoods is a common problem when drying temperatures are too high (Fig. 12–14). A deep grayish-brown chemical discoloration can occur in many hardwood species if initial drying is too slow (Fig. 12–15).

Fungal stains, often referred to as blue or sap stain, are caused by fungi that grow in the sapwood (Fig. 12–16). Blue-stain fungi do not cause decay of the sapwood, and fungi generally do not grow in heartwood. Blue stain can develop if initial drying is too slow.

Another common type of stain develops under stickers (Fig. 12–17). This stain results from contact of the sticker with the board. Sticker stains (sometimes called shadow) are imprints of the sticker that are darker or lighter than the wood between the stickers and can be caused by either chemical or fungal action, or both.

Moisture Content of Dried Lumber

Although widely used, the trade terms “shipping dry,” “air dry,” and “kiln dry” may not have identical meanings as to moisture content in the different producing regions. Despite the wide variations in the use of these terms, they are sometimes used to describe dried lumber. The following statements, which are not exact definitions, outline these categories.

Shipping Dry

Shipping dry means lumber that has been partially dried to prevent stain or mold during brief periods of transit; ideally the outer 3.2 mm (1/8 in.) is dried to 25% or less moisture content.

Table 12–3. Typical dry kiln schedules for lumber

Moisture content-based schedule for 25-mm (1-in.) (4/4) black walnut, dried to 7% moisture content

Moisture content (%)	Temperature (°C(°F)) Dry-bulb	Temperature (°C(°F)) Wet-bulb	Relative humidity (%)	Equilibrium moisture content (%)
Above 50	49.0 (120)	45.0 (113)	80	14.4
50 to 40	49.0 (120)	43.5 (110)	72	12.1
40 to 35	49.0 (120)	40.5 (105)	60	9.6
35 to 30	49.0 (120)	35.0 (95)	40	6.5
30 to 25	54.5 (130)	32.0 (90)	22	4.0
25 to 20	60.0 (140)	32.0 (90)	15	2.9
20 to 15	65.5 (150)	37.5 (100)	18	3.2
15 to 7	82.2 (180)	54.5 (130)	26	3.5
Equalize	82.2 (180)	58.3 (137)	30	3.8
Condition	82.2 (180)	76.7 (170)	79	11.1

Time-based schedule for 25- to 50-mm (1- to 2-in.) (4/4 to 8/4) Douglas Fir, upper grades, dried to 12% moisture content

Time (h)	Temperature (°C(°F)) Dry-bulb	Temperature (°C(°F)) Wet-bulb	Relative humidity (%)	Equilibrium moisture content (%)
0 to 12	76.5 (170)	73.5 (164)	86	14.1
12 to 24	76.5 (170)	71.0 (160)	78	11.4
24 to 48	79.5 (175)	71.0 (160)	69	9.1
48 to 72	82.2 (180)	71.0 (160)	62	7.7
72 to 96 or until dry	82.2 (180)	60.0 (140)	36	4.5

High temperature schedule for 50- by 100-mm to 50- by 250-mm (2- by 4-in. to 2- by 10-in.) Southern Pine, dried to 15% moisture content

Time (h)	Temperature (°C(°F)) Dry-bulb	Temperature (°C(°F)) Wet-bulb	Relative humidity (%)	Equilibrium moisture content (%)
0 until dry	116 (240)	82.2 (180)	29	2.5

Air Dry

Air dry means lumber that has been dried by exposure to the air outdoors or in a shed or by forced circulation of air that has not been heated above 49°C (120°F). Commercial air-dry stock generally has an average moisture content low enough for rapid kiln drying or rough construction use. Moisture content is generally in the range of 20% to 25% for dense hardwoods and 15% to 20% for softwoods and low-density hardwoods. Extended exposure can bring standard 19- and 38-mm (nominal 1- and 2-in.) lumber within one or two percentage points of the average exterior equilibrium moisture content of the region. For much of the United States, the minimum moisture content of thoroughly air-dried lumber is 12% to 15%.

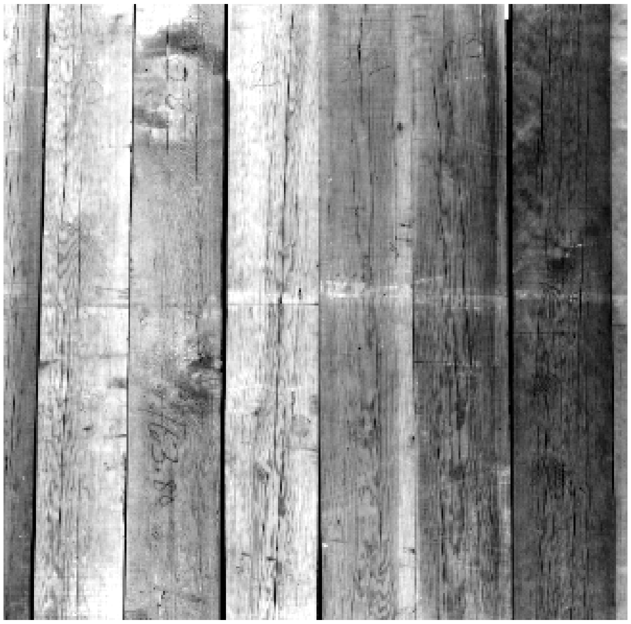

Figure 12–8. Surface checking on Douglas Fir dimension lumber.

Figure 12–9. End checking in oak lumber.

Figure 12–10. Severe collapse in western redcedar.

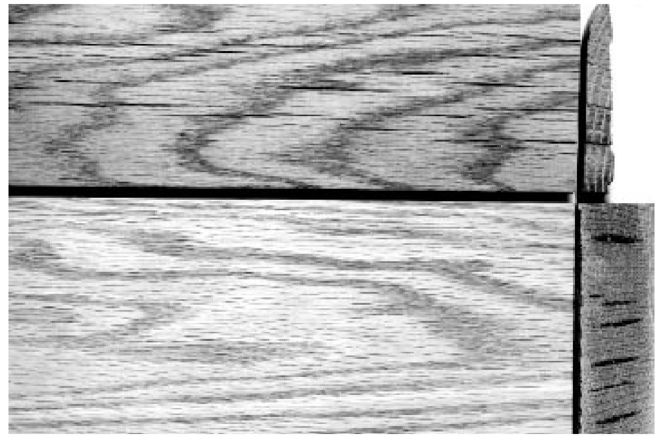

Figure 12–11. Board machined into millwork shows honeycomb (top). Surface of planed red oak board shows no honeycomb (bottom).

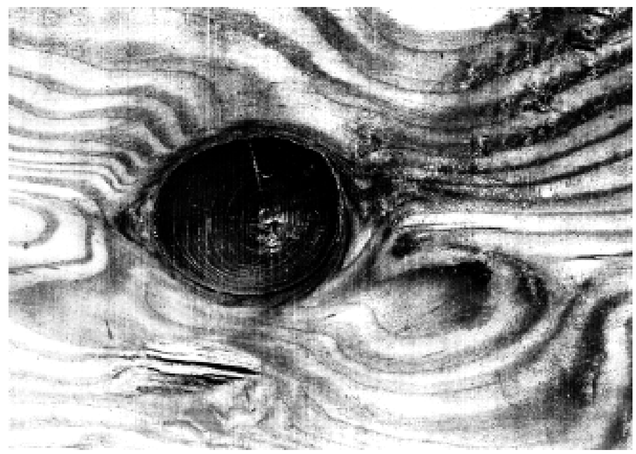

Figure 12–12. Loose knot in Southern Pine.

Kiln Dry

Kiln dry means lumber that has been dried in a kiln or by some special drying method to an average moisture content specified or understood to be suitable for a certain use. The average moisture content should have upper and lower tolerance limits, and all values should fall within these limits. Kiln-dried softwood dimension lumber generally has an average moisture content of 19% or less; the average moisture content for many other softwood uses is 10% to 20%. Hardwood and softwood lumber for furniture, cabinetry, and millwork usually has a final moisture content of 6% to 8% and can be specified to be free of drying stresses. The importance of suitable moisture content values is recognized, and provisions covering them are now incorporated in some softwood standards as grading rules. Moisture content values in the general grading rules may or may not be suitable for a specific use; if not, a special moisture content specification should be made.

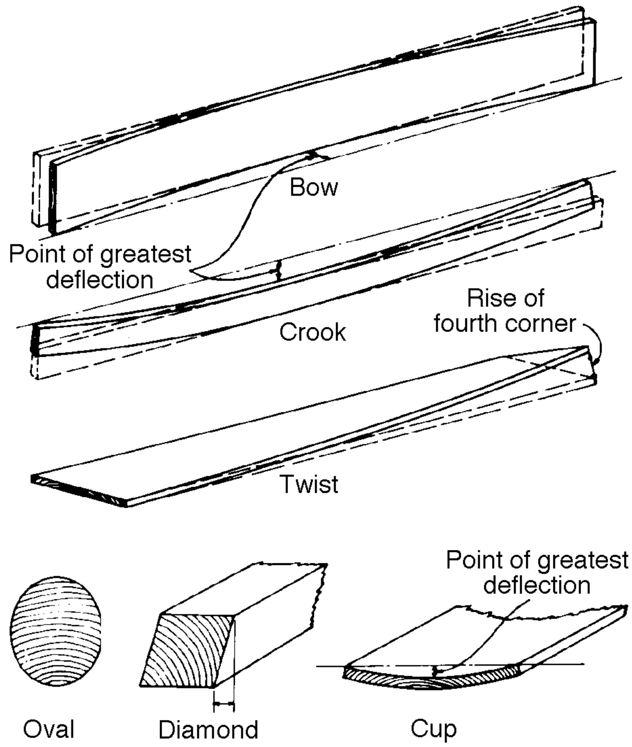

Figure 12–13. Various types of warp that can develop in boards during drying.

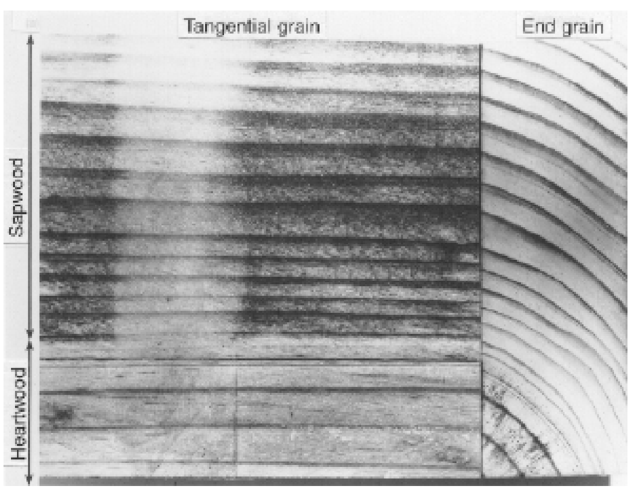

Figure 12–14. Brown sapwood stain in Southern Pine lumber.

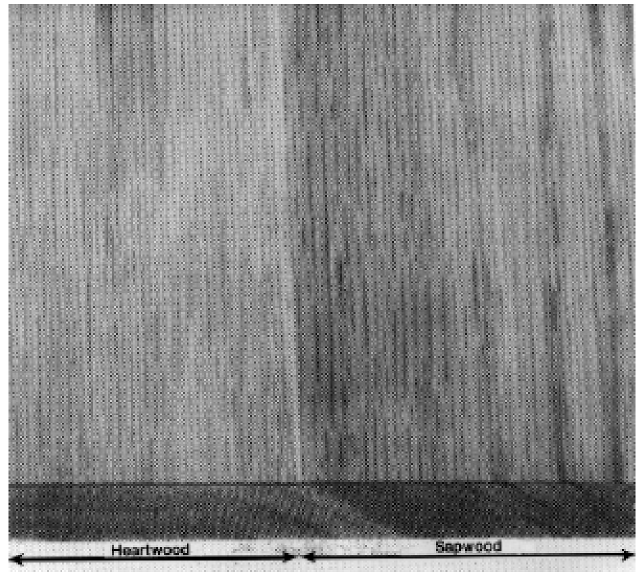

Figure 12–15. Gray sapwood stain in southern red oak that was dried green with humid, low temperature conditions and poor air circulation.

Figure 12–16. Sap stain in Southern Pine. Color ranges from bluish gray to black.

Figure 12–17. Sticker stain in sapwood of sugar maple after planing.

Moisture Control During Transit and Storage

Lumber and other wood items may change in moisture content and dimension while awaiting shipment, during fabrication, in transit, and in storage.

When standard 19-mm (nominal 1-in.) dry softwood lumber is shipped in tightly closed boxcars, shipping containers, or trucks or in packages with complete and intact wrappers, average moisture content changes for a package can generally be held to 0.2% or less per month. In holds or between decks of ships, dry material usually adsorbs about 1.5% moisture during normal shipping periods. If green material is included in the cargo, the moisture regain of the dry lumber may be doubled. On the top deck, if unprotected from the elements, the moisture regain can be as much as 7%.

When standard 19-mm (nominal 1-in.) softwood lumber, kiln dried to 8% or less, is piled solid under a good pile roof in a yard in humid weather, average moisture content of a pile can increase at the rate of about 2% per month during the first 45 days. An absorption rate of about 1% per month can then be sustained throughout a humid season. Comparable initial and sustaining absorption rates are about 1% per month in open (roofed) sheds and 0.3% per month in closed sheds. Stock that was piled for a year in an open shed in a western location increased 2.7% on the inside of solid piles and 3.5% on the outside of the piles. All stock that has been manufactured in any way should be protected from precipitation and spray, because water that gets into a solid pile tends to be absorbed by the wood instead of evaporating. The extent to which additional control of the storage environment is required depends upon the use to which the wood will be put and the corresponding moisture content recommendations. The moisture content of all stock should be determined when it is received. If moisture content is not as specified or required, stickered storage in an appropriate condition could ultimately bring the stock within the desired moisture content range. If a large degree of moisture change is required, the stock must be redried.

Plywood and Structural Items

Green or partially dried lumber and timbers should be open piled on stickers and protected from sunshine and precipitation by a tight roof. Framing lumber and plywood with 20% or less moisture content can be solid piled in a shed that provides good protection against sunshine and direct or wind-driven precipitation. However, a better practice for stock with greater than 12% moisture content is the use of stickered piling to bring moisture content more in line with the moisture content in use. Dry lumber can be piled solid in the open for relatively short periods, but at least a minimum pile cover of waterproofed paper should be used whenever possible. Because it is difficult to keep rain out completely, storing solid-piled lumber in the open for long periods is not recommended. If framing lumber must be stored in the open for a long time, it should be piled on stickers over good supports and the piles should be roofed. Solid-piled material that has become wet again should also be re-piled on stickers.

Table 12–4. Amount by which temperature of storage area must be increased above outside temperature to maintain equilibrium moisture content

Outside relative humidity (%)	Temperature differential (°C (°F)) for desired equilibrium moisture content						
	6%	7%	8%	9%	10%	11%	12%
90	18.3 (33)	16.1 (29)	12.8 (23)	10.0 (18)	8.3 (15)	6.1 (11)	5.0 (9)
80	16.7 (30)	13.9 (25)	10.5 (19)	7.8 (14)	6.1 (11)	4.4 (8)	3.3 (6)
70	13.9 (25)	11.1 (20)	8.3 (15)	5.6 (10)	3.9 (7)	2.2 (4)	1.7 (3)
60	11.1 (20)	8.3 (15)	5.0 (9)	3.3 (6)	1.7 (3)	—	—
50	8.3 (15)	5.6 (10)	2.8 (5)	0.6 (1)	—	—	—

Finish and Factory Lumber

Such kiln-dried items as exterior finish, siding, and exterior millwork should be stored in a closed but unheated shed. They should be placed on supports raised above the floor, at least 150 mm (6 in.) high if the floor is paved or 300 mm (12 in.) if not paved. Interior trim, flooring, cabinet work, and lumber for processing into furniture should be stored in a room or closed shed where relative humidity is controlled. Kiln-dried and machined hardwood dimension or softwood cut stock should also be stored under controlled humidity conditions.

Dried and machined hardwood dimension or softwood lumber intended for remanufacture should also be stored under controlled humidity conditions. Under uncontrolled conditions, the ends of such stock may attain a greater moisture content than the balance of the length. Then, when the stock is straight-line ripped or jointed before edge gluing, subsequent shrinkage will cause splitting or open glue joints at the ends of panels. The simplest way to reduce relative humidity in storage areas of all sizes is to heat the space to a temperature slightly greater than that of the outside air. Dehumidifiers can be used in small, well-enclosed spaces.

If the heating method is used, and there is no source of moisture except that contained in the air, the equilibrium moisture content can be maintained by increasing the temperature of the storage area greater than the outside temperature by the amounts shown in Table 12–4. When a dehumidifier is used, the average temperature in the storage space should be known or controlled. Table 3–4 in Chapter 3 should be used to select the proper relative humidity to give the desired average moisture content. Wood in a factory awaiting or following manufacture can become too dry if the area is heated to 21°C (70°F) or greater when the outdoor temperature is low. This often occurs in the northern United States during the winter. Under such circumstances, exposed ends and surfaces of boards or cut pieces will tend to dry to the low equilibrium moisture content condition, causing shrinkage and warp. In addition, an equilibrium moisture content of 4% or more below the moisture content of the core of freshly crosscut boards can cause end checking. Simple remedies are to cover piles of partially manufactured items with plastic film and lower the shop temperature during nonwork hours. Increased control can be obtained in critical shop and storage areas by humidification. In warm weather, cooling can increase relative humidity and dehumidification may be necessary.

Dimensional Changes in Wood

Dry wood undergoes small changes in dimension with normal changes in relative humidity. More humid air will cause slight swelling, and drier air will cause slight shrinkage. These changes are considerably smaller than those involved with shrinkage from the green condition. Equation (12–2) can be used to approximate dimensional changes caused by shrinking and swelling by using the total shrinkage coefficient from green to ovendry. However, the equation assumes that the shrinkage–moisture content relationship is linear. Figure 3–4 (Ch. 3) shows that this is not the case, so some error is introduced. The error is in the direction of underestimating dimensional change, by about 5% of the true change. Many changes of moisture content in use are over the small moisture content range of 6% to 14%, where the shrinkage–moisture content relationship is linear (Ch. 3, Fig. 3–4). Therefore, a set of shrinkage coefficients based on the linear portion of the shrinkage–moisture content curve has been developed (Table 12–5). Approximate changes in dimension can be estimated by a simple formula that involves a dimensional change coefficient, from Table 12–5, when moisture content remains within the range of normal use. (Dimensional changes are further discussed in Chs. 3 and 6.)

Estimation Using Dimensional Change Coefficient

The change in dimension within the moisture content limits of 6% to 14% can be estimated satisfactorily by using a dimensional change coefficient based on the dimension at 10% moisture content:

$$\Delta D = D_{\mathrm{I}}\left[C_{\mathrm{T}}(M_{\mathrm{F}} - M_{\mathrm{I}})\right] \qquad (12\text{–}2)$$

where ΔD is change in dimension, D_{I} dimension in units of length at start of change, C_{T} dimensional change coefficient tangential direction (for radial direction, use C_{R}), M_{F} moisture content (%) at end of change, and M_{I} moisture content (%) at start of change.

Values for C_{T} and C_{R}, derived from total shrinkage values, are given in Table 12–5. When $M_{\mathrm{F}} < M_{\mathrm{I}}$, the quantity ($M_{\mathrm{F}} - M_{\mathrm{I}}$) will be negative, indicating a decrease in dimension; when greater, it will be positive, indicating an increase in dimension.

As an example, assuming the width of a flat-grained white fir board is 232 mm (9.15 in.) at 8% moisture content, its change in width at 11% moisture content is estimated as

$$\begin{aligned}\Delta D &= 232[0.00245(11 - 8)]\\ &= 232(0.00735)\\ &= 1.705 \text{ mm}\\ \Delta D &= 9.15[0.00245(11 - 8)]\\ &= 9.15[0.00735]\\ &= 0.06725 \text{ or } 0.067 \text{ in.}\end{aligned}$$

Then, dimension at end of change

$$\begin{aligned}D_{\mathrm{I}} + \Delta D &= 232 + 1.7 && (= 9.15 + 0.067)\\ &= 233.7 \text{ mm} && (= 9.217 \text{ in.})\end{aligned}$$

The thickness of the same board at 11% moisture content can be estimated by using the coefficient $C_{\mathrm{R}} = 0.00112$.

Table 12–5. Coefficients for dimensional change as a result of shrinking or swelling within moisture content limits of 6% to 14% (C_T = dimensional change coefficient for tangential direction; C_R = radial direction)

Species	Dimensional change coefficient[a] C_R	C_T	Species	Dimensional change coefficient[a] C_R	C_T
Hardwoods					
Alder, red	0.00151	0.00256	Honeylocust	0.00144	0.00230
Apple	0.00205	0.00376	Locust, black	0.00158	0.00252
Ash, black	0.00172	0.00274	Madrone, Pacific	0.00194	0.00451
Ash, Oregon	0.00141	0.00285	Magnolia, cucumbertree	0.00180	0.00312
Ash, pumpkin	0.00126	0.00219	Magnolia, southern	0.00187	0.00230
Ash, white	0.00169	0.00274	Magnolia, sweetbay	0.00162	0.00293
Ash, green	0.00169	0.00274	Maple, bigleaf	0.00126	0.00248
Aspen, quaking	0.00119	0.00234	Maple, red	0.00137	0.00289
Basswood, American	0.00230	0.00330	Maple, silver	0.00102	0.00252
Beech, American	0.00190	0.00431	Maple, black	0.00165	0.00353
Birch, paper	0.00219	0.00304	Maple, sugar	0.00165	0.00353
Birch, river	0.00162	0.00327	Oak, black	0.00123	0.00230
Birch, yellow	0.00256	0.00338	Red Oak, commercial	0.00158	0.00369
Birch, sweet	0.00256	0.00338	Red oak, California	0.00123	0.00230
Buckeye, yellow	0.00123	0.00285	Red oak: water, laurel, willow	0.00151	0.00350
Butternut	0.00116	0.00223	White Oak, commercial	0.00180	0.00365
Catalpa, northern	0.00085	0.00169	White oak, live	0.00230	0.00338
Cherry, black	0.00126	0.00248	White oak, Oregon white	0.00144	0.00327
Chestnut, American	0.00116	0.00234	White oak, overcup	0.00183	0.00462
Cottonwood, black	0.00123	0.00304	Persimmon, common	0.00278	0.00403
Cottonwood, eastern	0.00133	0.00327	Sassafras	0.00137	0.00216
Elm, American	0.00144	0.00338	Sweet gum	0.00183	0.00365
Elm, rock	0.00165	0.00285	Sycamore, American	0.00172	0.00296
Elm, slippery	0.00169	0.00315	Tanoak	0.00169	0.00423
Elm, winged	0.00183	0.00419	Tupelo, black	0.00176	0.00308
Elm, cedar	0.00183	0.00419	Tupelo, water	0.00144	0.00267
Hackberry	0.00165	0.00315	Walnut, black	0.00190	0.00274
Hickory, pecan	0.00169	0.00315	Willow, black	0.00112	0.00308
Hickory, true	0.00259	0.00411	Willow, Pacific	0.00099	0.00319
Holly, American	0.00165	0.00353	Yellow-poplar	0.00158	0.00289
Softwoods					
Baldcypress	0.00130	0.00216	Pine, eastern white	0.00071	0.00212
Cedar, yellow	0.00095	0.00208	Pine, jack	0.00126	0.00230
Cedar, Atlantic white	0.00099	0.00187	Pine, loblolly	0.00165	0.00259
Cedar, eastern red	0.00106	0.00162	Pine, pond	0.00165	0.00259
Cedar, Incense	0.00112	0.00180	Pine, lodgepole	0.00148	0.00234
Cedar, Northern white[b]	0.00101	0.00229	Pine, Jeffrey	0.00148	0.00234
Cedar, Port-Orford	0.00158	0.00241	Pine, longleaf	0.00176	0.00263
Cedar, western red[b]	0.00111	0.00234	Pine, ponderosa	0.00133	0.00216
Douglas-fir, Coast-type	0.00165	0.00267	Pine, red	0.00130	0.00252
Douglas-fir, Interior north	0.00130	0.00241	Pine, shortleaf	0.00158	0.00271
Douglas-fir, Interior west	0.00165	0.00263	Pine, slash	0.00187	0.00267
Fir, balsam	0.00099	0.00241	Pine, sugar	0.00099	0.00194
Fir, California red	0.00155	0.00278	Pine, Virginia	0.00144	0.00252
Fir, noble	0.00148	0.00293	Pine, western white	0.00141	0.00259

Table 12–5. Coefficients for dimensional change as a result of shrinkage or swelling within moisture content limits of 6% to 14% (C_T = dimensional change coefficient for tangential direction; C_R = radial direction)—con.

Species	Dimensional change coefficient[a]		Species	Dimensional change coefficient[a]	
	C_R	C_T		C_R	C_T
Softwoods—con.					
Fir, Pacific silver	0.00151	0.00327	Redwood, old-growth[b]	0.00120	0.00205
Fir, subalpine	0.00088	0.00259	Redwood, second-growth[b]	0.00101	0.00229
Fir, grand	0.00112	0.00245	Spruce, black	0.00141	0.00237
Fir, white	0.00112	0.00245	Spruce, Engelmann	0.00130	0.00248
Hemlock, eastern	0.00102	0.00237	Spruce, red	0.00130	0.00274
Hemlock, western	0.00144	0.00274	Spruce, white	0.00130	0.00274
Larch, western	0.00155	0.00323	Spruce, Sitka	0.00148	0.00263
			Tamarack	0.00126	0.00259
Imported Woods					
Andiroba, crabwood	0.00137	0.00274	Light red “Philippine mahogany”	0.00126	0.00241
Angelique	0.00180	0.00312	Limba	0.00151	0.00187
Apitong, keruing[b] (all *Dipterocarpus* spp.)	0.00243	0.00527	Mahogany[b]	0.00172	0.00238
			Meranti	0.00126	0.00289
Avodire	0.00126	0.00226	Obeche	0.00106	0.00183
Balsa	0.00102	0.00267	Okoume	0.00194	0.00212
Banak	0.00158	0.00312	Parana, pine	0.00137	0.00278
Cativo	0.00078	0.00183	Paumarfim	0.00158	0.00312
Cuangare	0.00183	0.00342	Primavera	0.00106	0.00180
Greenheart[b]	0.00390	0.00430	Ramin	0.00133	0.00308
Iroko[b]	0.00153	0.00205	Santa Maria	0.00187	0.00278
Khaya	0.00141	0.00201	Spanish-cedar	0.00141	0.00219
Kokrodua[b]	0.00148	0.00297	Teak[b]	0.00101	0.00186
Lauans: dark red “Philippine mahogany”	0.00133	0.00267			

[a]Per 1% change in moisture content, based on dimension at 10% moisture content and a straight-line relationship between moisture content at which shrinkage starts and total shrinkage. (Shrinkage assumed to start at 30% for all species except those indicated by footnote b.)
[b]Shrinkage assumed to start at 22% moisture content.

Because commercial lumber is often not perfectly flatsawn or quartersawn, this procedure will probably overestimate width shrinkage and underestimate thickness shrinkage. Note also that if both a size change and the percentage of moisture content are known, Equation (12–2) can be used to calculate the original moisture content.

Calculation Based on Green Dimensions

Approximate dimensional changes associated with moisture content changes greater than 6% to 14% , or when one moisture value is outside of those limits, can be calculated by

$$\Delta D = \frac{D_{\mathrm{I}}(M_{\mathrm{F}} - M_{\mathrm{I}})}{30(100)/S_{\mathrm{T}} - 30 + M_{\mathrm{I}}} \qquad (12\text{–}3)$$

where S_T is tangential shrinkage (%) from green to ovendry (Ch. 3, Tables 3–5 and 3–6) (use radial shrinkage S_R when appropriate).

Neither M_I nor M_F should exceed 30%, the assumed moisture content value when shrinkage starts for most species.

Design Factors Affecting Dimensional Change

Framing Lumber in House Construction

Ideally, house framing lumber should be dried to the moisture content it will reach in use, thus minimizing future dimensional changes as a result of frame shrinkage. This ideal condition is difficult to achieve, but some drying and shrinkage of the frame may take place without being visible or causing serious defects after the house is completed. If, at the time the wall and ceiling finish is applied, the moisture content of the framing lumber is not more than about 5% above that which it will reach in service, there will be little or no evidence of defects caused by shrinkage of the frame. In heated houses in cold climates, joists over heated basements, studs, and ceiling joists may reach a moisture content as low as 6% to 7% (Table 12–2). In mild climates, the minimum moisture content will be greater.

The most common signs of excessive shrinkage are cracks in plastered walls, truss rise, open joints, and nail pops in drywall construction; distortion of door openings; uneven floors; and loosening of joints and fastenings. The extent of vertical shrinkage after the house is completed is proportional to the depth of wood used as supports in a horizontal position, such as girders, floor joists, and plates. After all, shrinkage occurs primarily in the width of members, not the length.

Thorough consideration should be given to the type of framing best suited to the whole building structure. Methods should be chosen that will minimize or balance the use of wood across the grain in vertical supports. These involve variations in floor, wall, and ceiling framing. The factors involved and details of construction are covered extensively in *Wood-Frame House Construction* (Sherwood and Stroh 1991).

Heavy Timber Construction

In heavy timber construction, a certain amount of shrinkage is to be expected. A column that bears directly on a wood girder can result in a structure settling as a result of the perpendicular-to-grain shrinkage of the girder. If not provided for in the design, shrinkage may cause weakening of the joints or uneven floors or both. One means of eliminating part of the shrinkage in mill buildings and similar structures is to use metal post caps; the metal in the post cap separates the upper column from the lower column. The same thing is accomplished by bolting wood corbels to the side of the lower column to support the girders.

When joist hangers are installed, the top of the joist should be above the top of the girder; otherwise, when the joist shrinks in the stirrup, the floor over the girder will be higher than that bearing upon the joist. Heavy planking used for flooring should be near 12% moisture content to minimize openings between boards as they approach moisture equilibrium. When standard 38- or 64-mm (nominal 2- or 3-in.) joists are nailed together to provide a laminated floor of greater depth for heavy design loads, the joist material should be somewhat less than 12% moisture content if the building is to be heated.

Interior Finish

The normal seasonal changes in the moisture content of interior finish are not enough to cause serious dimensional change if the woodwork is carefully designed. Large members, such as ornamental beams, cornices, newel posts, stair stringers, and handrails, should be built up from comparatively small pieces. Wide door and window trim and base should be hollow-backed. Backband trim, if mitered at the corners, should be glued and splined before erection; otherwise butt joints should be used for the wide faces. Large, solid pieces, such as wood paneling, should be designed and installed so that the panels are free to move across the grain. Narrow widths are preferable.

Flooring

Flooring is usually dried to the moisture content expected in service so that shrinking and swelling are minimized and buckling or large gaps between boards do not occur. For basement, large hall, or gymnasium floors, however, enough space should be left around the edges to allow for some expansion.

Wood Care and Installation During Construction

Lumber and Trusses

Although it should be, lumber is often not protected from the weather at construction sites. Lumber is commonly placed on the ground in open areas near the building site as bulked and strapped packages. Supports under such packages are useful to prevent wetting from mud and ground water and should elevate the packages at least 150 mm (6 in.) off the ground. The packages should also be covered with plastic tarpaulins for protection from rain.

Lumber that is green or nearly green should be piled in stickers under a roof for additional drying before it is built into the structure. The same procedure is required for lumber that has been treated with a waterborne preservative but not fully redried. Prefabricated building parts, such as roof trusses, sometimes lie unprotected on the ground at the building site. In warm, rainy weather, moisture regain can result in fungal staining. Wetting of the lumber also results in swelling, and subsequent shrinkage of the framing may contribute to structural distortions. Extended storage of lumber at moisture contents greater than 20% without drying can allow decay to develop.

If framing lumber has a greater moisture content when installed than that recommended in Table 12–2, shrinkage can be expected. Framing lumber, even thoroughly air-dried stock, will generally have a moisture content greater than that recommended when it is delivered to the building site. If carelessly handled in storage at the site, the lumber can take up more moisture. Builders can schedule their work so an appreciable amount of drying can take place during the early stages of construction. This minimizes the effects of additional drying and shrinkage after completion. When the house has been framed, sheathed, and roofed, the framing is so exposed that in time it can dry to a lower moisture content than would ordinarily be expected in yard-dried lumber. The application of the wall and ceiling finish is delayed while wiring and plumbing are installed. If this delay is about 30 days in warm, dry weather, framing lumber should lose enough moisture so that any additional drying in place will be relatively unimportant. In cool, damp weather, or if wet lumber is used, the period of exposure should be extended. Checking moisture content of door and window headers and floor and ceiling joists at this time with an electric moisture meter is good practice. When these members approach an average of 12% moisture content, interior finish and trim can normally be installed. Closing the house and using the heating system will hasten the rate of drying.

Before wall finish is applied, the frame should be examined and defects that may have developed during drying, such as warped or distorted studs, shrinkage of lintels over openings, or loosened joints, should be corrected.

Exterior Trim and Millwork

Exterior trim, such as cornice and rake mouldings, fascia boards, and soffit material, is normally installed before the shingles are laid. Trim, siding, and window and door frames should be protected on the site by storing in the house or garage until time of installation. Although items such as window frames and sashes are usually treated with some type of water-repellent preservative to resist absorption of water, they should be stored in a protected area if they cannot be installed soon after delivery. Wood siding is often received in packaged form and can ordinarily remain in the package until installation.

Finished Flooring

Cracks develop in flooring if it absorbs moisture either before or after it is laid, then shrinks when the building is heated. Such cracks can be greatly reduced by observing the following practices:

- Specify flooring manufactured according to association rules and sold by dealers that protect it properly during storage and delivery.
- Do not allow flooring to be delivered before masonry and plastering are completed and fully dry, unless a dry storage space is available.
- Install the heating plant before flooring is delivered.
- Break open flooring bundles and expose all sides of flooring to the atmosphere inside the structure.
- Close up the house at night and increase the temperature about 8°C (15°F) greater than the outdoor temperature for about 3 days before laying the floor.
- If the house is not occupied immediately after the floor is laid, keep the house closed at night or during damp weather and supply some heat if necessary.

Better and smoother sanding and finishing can be done when the house is warm and the wood has been kept dry.

Interior Finish

In a building under construction, average relative humidity will be greater than that in an occupied house because of the moisture that evaporates from wet concrete, brickwork, plaster, and even the structural wood members. The average temperature will be lower because workers prefer a lower temperature than is common in an occupied house. Under such conditions, the finish tends to have greater moisture content during construction than it will have during occupancy.

Before the interior finish is delivered, the outside doors and windows should be hung in place so that they can be kept closed at night. In this way, conditions of the interior can be held as close as possible to the higher temperature and lower humidity that ordinarily prevail during the day. Such protection may be sufficient during dry warm weather, but during damp or cool weather, it is highly desirable that some heat be maintained in the house, particularly at night. Whenever possible, the heating plant should be placed in the house before the interior trim is installed, to be available for supplying the necessary heat. Portable heaters can also be used. The temperature during the night should be maintained about 8°C (15°F) greater than the outside temperature but should not be allowed to drop below about 21°C (70°F) during the summer or 17°C (62°F) when the outside temperature is below freezing.

After buildings have thoroughly dried, less heat is needed, but unoccupied houses, new or old, should not be allowed to stand without some heat during the winter. A temperature of about 8°C (15°F) greater than the outside temperature and above freezing at all times will keep the woodwork, finish, and other parts of the house from being affected by dampness or frost.

Plastering

During a plastering operation in a moderate-sized, six-room house, approximately 450 kg (1,000 lb) of water are used, all of which must be dissipated before the house is ready for the interior finish. Adequate ventilation to remove the evaporated moisture will keep it from being absorbed by the framework. In houses plastered in cold weather, the excess moisture can also cause paint to blister on exterior finish and siding.

During warm, dry weather, with the windows wide open, the moisture will be gone within a week after the final coat of plaster is applied. During damp, cold weather, the heating system or portable heaters are used to prevent freezing of plaster and to hasten its drying. Adequate ventilation should be provided at all times of the year because a large volume of air is required to carry away the amount of water involved. Even in the coldest weather, the windows on the side of the house away from the prevailing winds should be opened 50 to 75 mm (2 to 3 in.), preferably from the top.

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