Chapter 11

Glued Structural Members

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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.

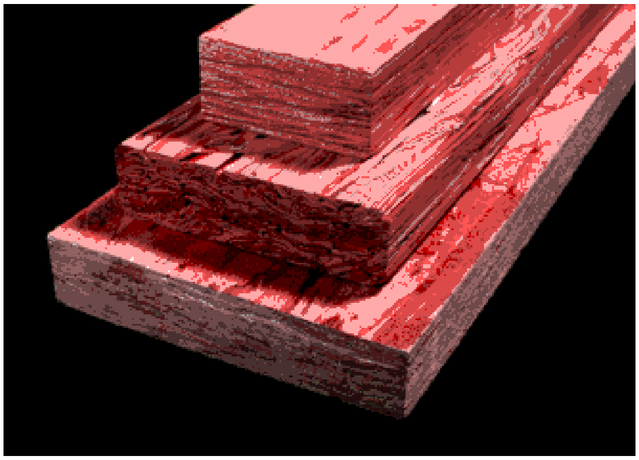

Figure 11–1. Examples of three types of SCL (top to bottom): laminated veneer lumber (LVL), parallel strand lumber (PSL), and oriented strand lumber (OSL).

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.

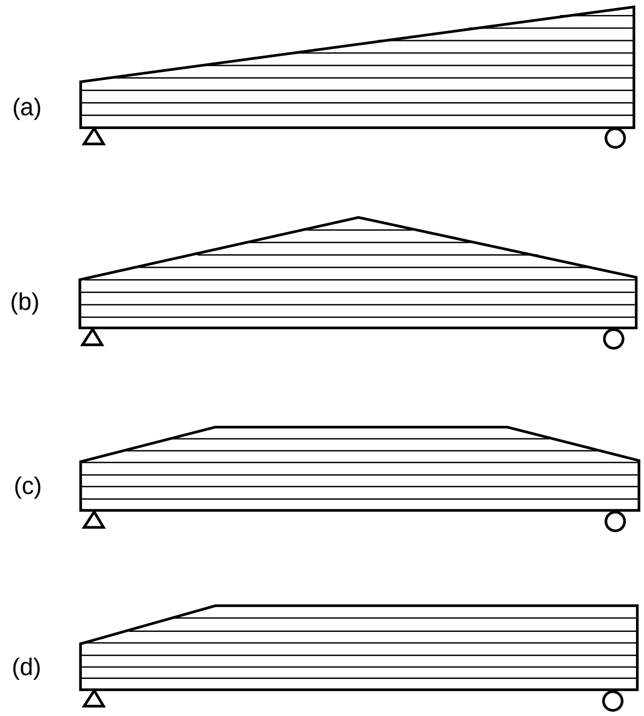

Figure 11–2. Glulam timbers may be (a) single tapered, (b) double tapered, (c) tapered at both ends, or (d) tapered at one end.

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.

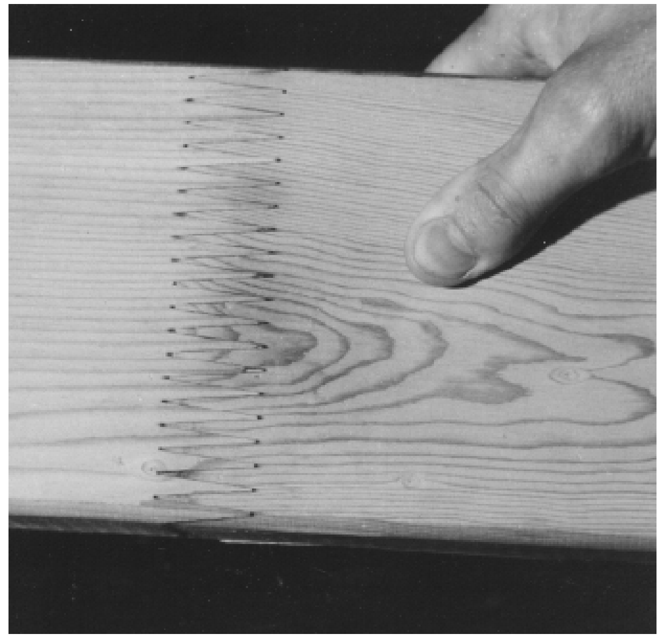

Figure 11–4. Typical fingerjoint used in the manufacture of glulam.

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.

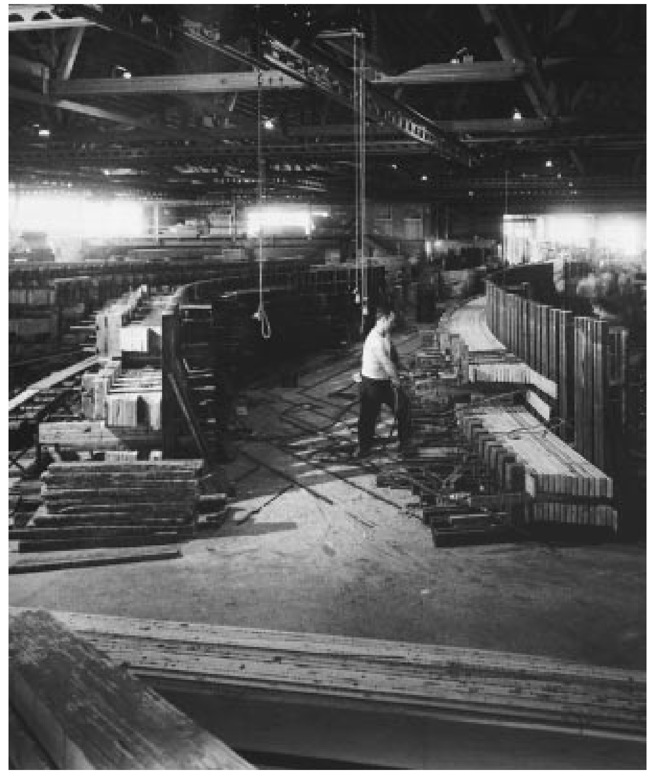

Figure 11–5. After being placed in the clamping bed, the laminations of these arches are forced together with an air-driven screw clamp.

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).

$$C_M = 1.0 \text{ for moisture content} \le 16\%$$

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.

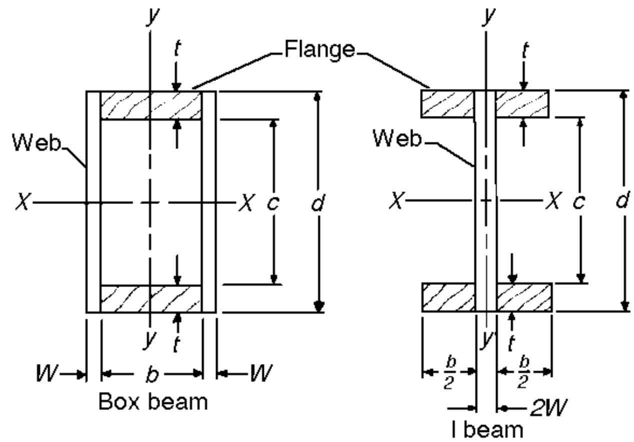

Figure 11–6. Beams with structural panel webs.

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

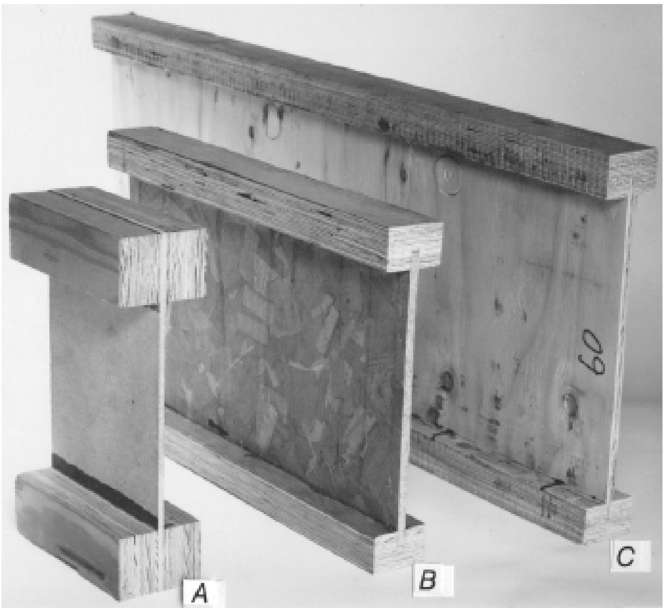

Figure 11–7. Prefabricated I-joists with laminated veneer lumber flanges and structural panel webs. (A) One experimental product has a hardboard web. The other two commercial products have (B) oriented strandboard and (C) plywood webs.

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

$$GA' = G s t_c \tag{11–15}$$

where G is stringer shear modulus.

Skin Stresses

Skin tensile and compressive stresses are given by

$$\begin{aligned} f_{x1} &= \frac{M E_1 y_1}{EI} \\ f_{x2} &= \frac{M E_2 y_2}{EI} \end{aligned} \tag{11–16}$$

where EI is given by Equation (11–14), M is bending moment, and

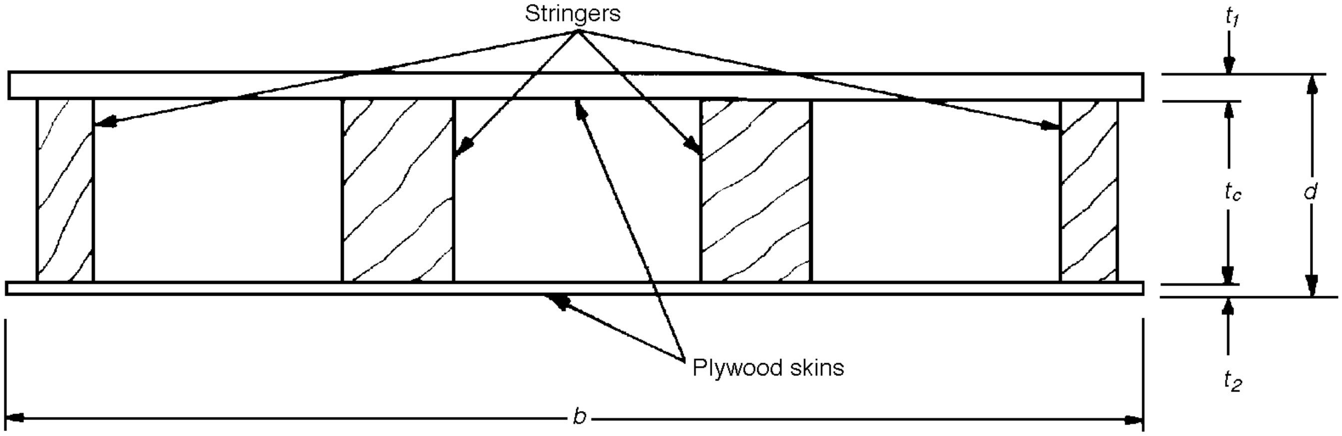

Figure 11–8. Stressed-skin panel cross section.

$$y_1 = \frac{E_2 t_2[(t_1 + t_c) + (t_2 + t_c)] + E t_c \frac{s}{b}(t_1 + t_c)}{2\left(E_1 t_1 + E_2 t_2 + E t_c \frac{s}{b}\right)}$$

$$y_2 = \frac{E_1 t_1[(t_1 + t_c) + (t_2 + t_c)] + E t_c \frac{s}{b}(t_2 + t_c)}{2\left(E_1 t_1 + E_2 t_2 + E t_c \frac{s}{b}\right)}$$

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

$$f_{sx1} = \frac{ME(y_1 - t_1/2)}{EI}$$
$$f_{sx2} = \frac{ME(y_2 - t_2/2)}{EI} \tag{11–17}$$

and these should not exceed appropriate values for the species.

The stringer shear stress is given by

$$f_{sxy} = \frac{V(EQ)}{sEI} \tag{11–18}$$

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

$$f_{gl} = \frac{V(EQ)}{sEI} \tag{11–19}$$

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

$$P_{cr} = \frac{\pi^2 EI}{L^2} \tag{11–20}$$

where L is unsupported panel length and EI is bending stiffness given by Equation (11–14).

Compressive stress in the skins is given by

$$f_{xc1} = \frac{PE_1}{EA}$$
$$f_{xc2} = \frac{PE_2}{EA} \tag{11–21}$$

and in the stringers by

$$f_{sxc} = \frac{PE}{EA} \tag{11–22}$$

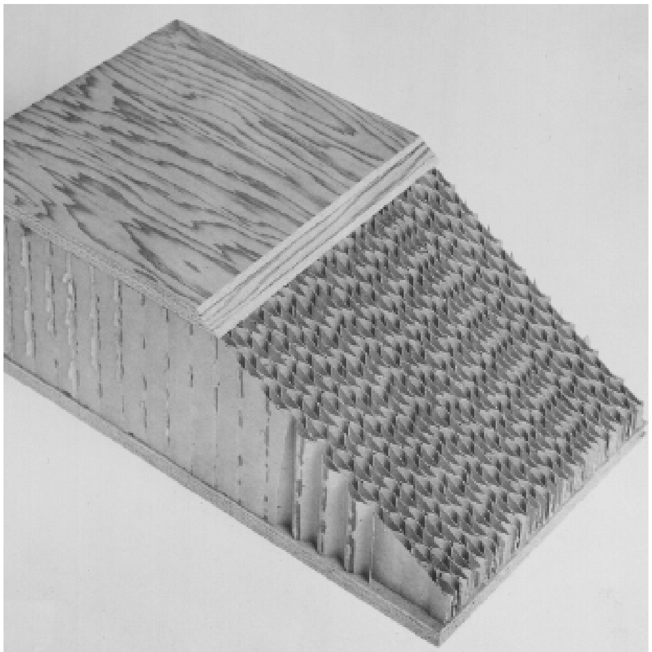

Figure 11–9. Cutaway section of sandwich construction with plywood facings and a paper honeycomb core.

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.

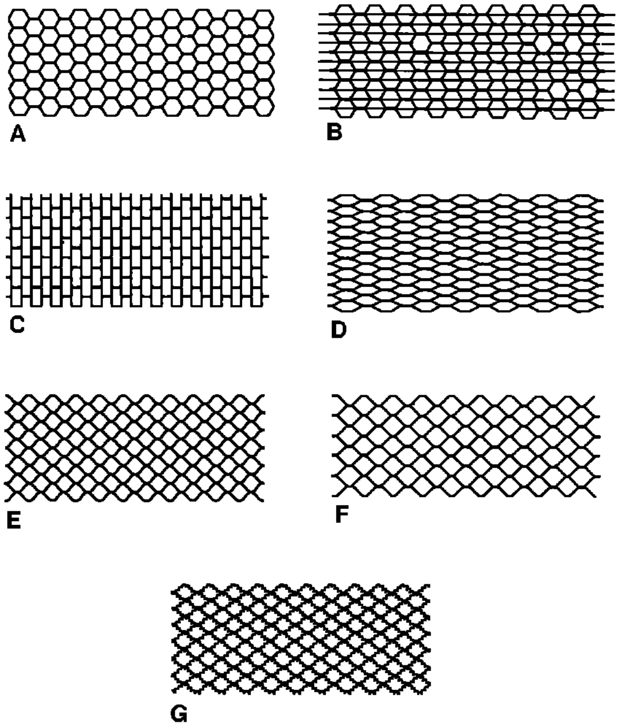

Figure 11–10. Honeycomb core cell configurations.

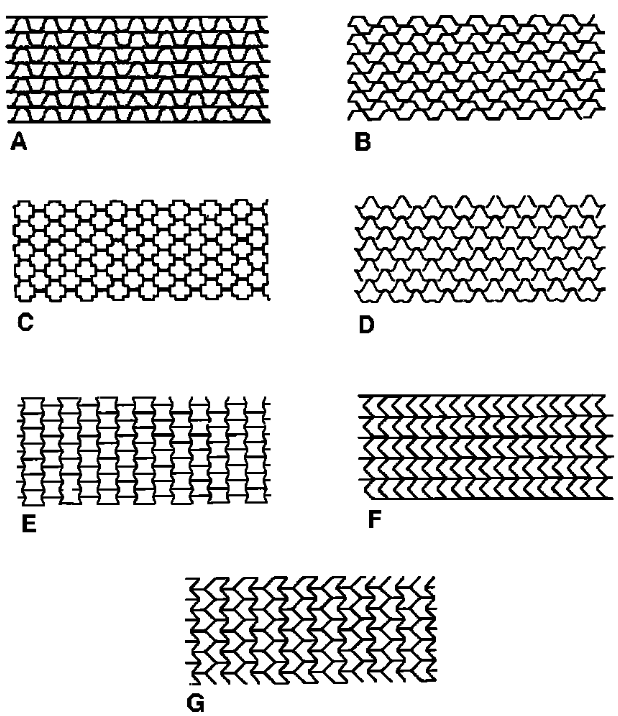

Figure 11–11. Cell configurations for formable paper honeycomb cores.

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

$$\frac{1}{N} = \frac{1}{N_{\mathrm{E}}} + \frac{1}{U} \tag{11–29}$$

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.

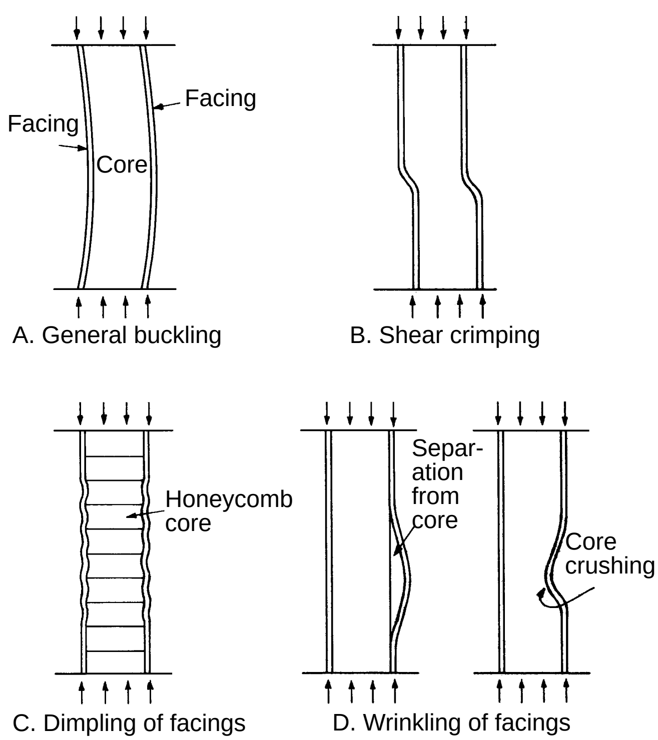

Figure 11–12. Modes of failure of sandwich construction under edgewise loads.

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

$$f = \frac{N}{t_1 + t_2} \tag{11–30}$$

In a strip of sandwich construction subjected to bending moments, the mean facing stresses are given by

$$f_i = \frac{M}{t_i h} \qquad i = 1,2 \tag{11–31}$$

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

$$f_{\mathrm{cs}} = \frac{S}{h} \tag{11–32}$$

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

$$f_{\mathrm{d}} = 2E\left(\frac{t_{\mathrm{f}}}{s}\right)^2 \tag{11–33}$$

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

$$f_{\mathrm{w}} = \frac{3}{4}(EE_{\mathrm{c}}G_{\mathrm{c}})^{1/3} \tag{11–34}$$

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.

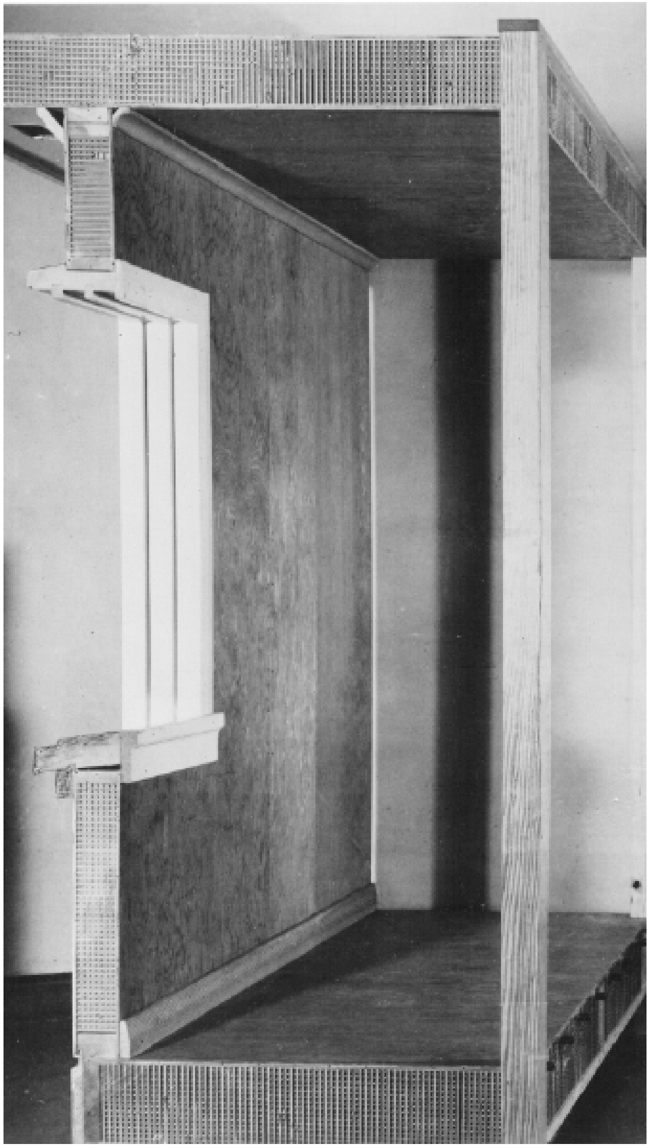

Figure 11–13. Cutaway to show details of sandwich construction in an experimental structure.

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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