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Experimental Shear Strength of Glued-Laminated Beams

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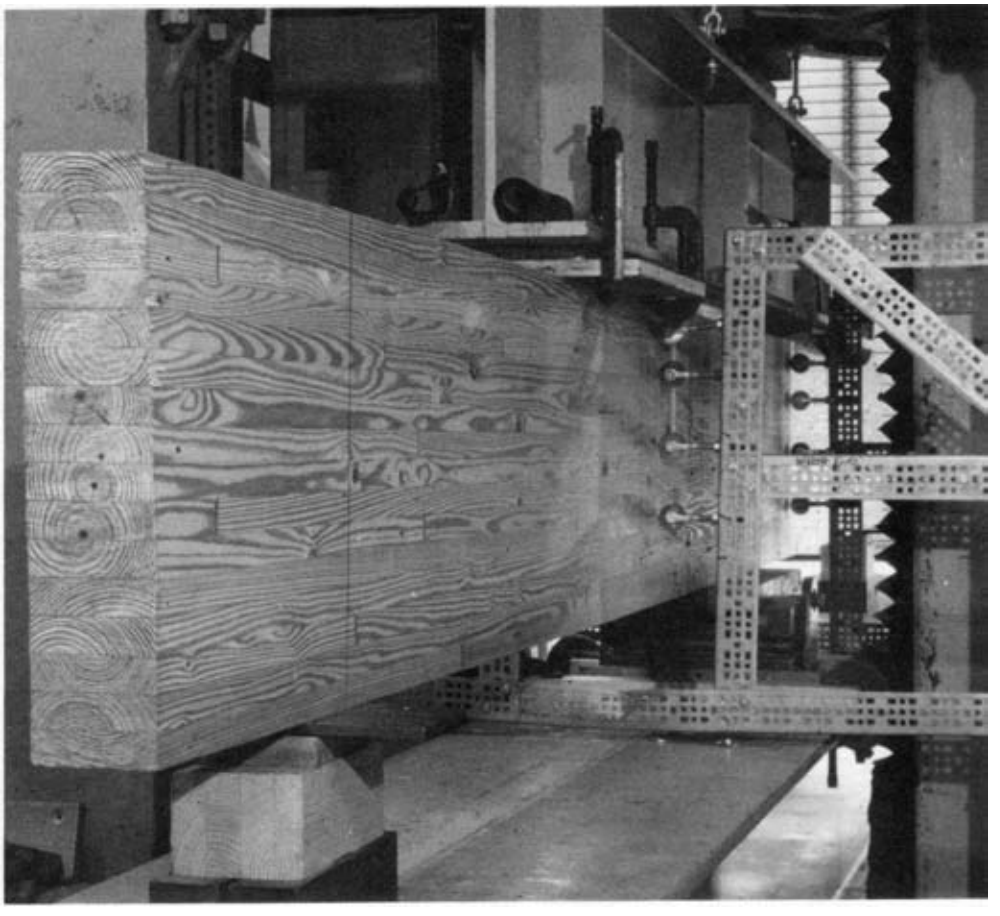

Abstract

Current shear design specifications ignore the effects of beam size. The ASTM D245 adjustment that relates ASTM D143 shear block results to design shear strength is unclear. An experimental study was conducted to determine shear and bending strength of matched Southern pine and Douglas-fir glued-laminated beams. A fivepoint loading setup determined beam shear strength values. These strength values were compared with the results of small. clear ASTM D143 specimens cut from the failed beam shear specimens. Bending strength was determined by third-point loading beam tests. Statistical methods of regression and analysis of variance investigated the following possible correlations: shear strength to beam size. shear strength to bending strength. and beam shear strength to ASTM shear block strength. Results from the study indicate that (1) a five-point test setup can consistently produce beam shear failures from a wide range of beam sues. (2) no apparent correlation exists between modulus of rupture and shear strength. (3) shear strength is dependent on beam size. and (4) beam shear strength is related to ASTM D143 shear strength values provided the reentrant comer stress-concentration effects are considered.

Keywords: Beams, design, shear strength, shear stress, modulus of rupture, shear tests, glued-laminated

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Experimental Shear Strength of Glued-Laminated Beams

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Introduction

Beam design is governed by either shear, bending, compression, or deflection criteria. Often the shear stress criteria will govern a beam design at a certain span-to-depth ratio; however, the beam will typically fail in bending during laboratory tests or field observations. This design inconsistency implies that shear design methods may be overly conservative.

Allowable shear design stresses are established in American Society for Testing and Materials D245 (ASTM 1993c). Ethington and others (1979) described the historical development of the ASTM D245 factors that relate small, clear shear block strength to allowable beam shear stress. Their observations are somewhat unclear on the origin or basis of factors. For example, they state that the duration-of-load factors were borrowed from bending results. Overload factors are "purely judgment," and for checks, "no evidence has been found to relate to the factor of 9/4." A more realistic relationship that relates clear, wood to beam shear strength could clarify or give credence to ASTM D245 factors. Historically, it has been difficult to experimentally create shear failures in wooden beams. This experimental inability resulted in the application of conservative adjustment factors for beam shear design. Therefore, to determine a realistic relationship between clear, wood and beam shear strength, a shear test method that is applicable to all beam sizes is needed.

Soltis and Gerhardt (1988) reviewed current shear design and made two conclusions. First, the accepted Newlin's two-beam theory for determining the stress in a split beam "incorrectly predicts shear stress distribution for unchecked beams." Second, further research is needed to clarify the effects of beam size, checks, and loading on shear strength. In general, a re-evaluation of current shear design methodology is needed.

The objective of this study was to improve current shear design criteria by (1) establishing a test method to determine beam shear. (2) establishing a database for beam shear strength and correlating it to shear block test results, (3) determining if there is a correlation between shear and bending strength, and (4) determining if there is a correlation between shear strength and beam size.

Background

Shear strength of wood has been investigated since the turn of the 20th century. Many experimental methods have been introduced to characterize shear strength. Examples are single shear, double shear, shear bobbin, torque tubes, and others. In the mid-1920s. ASTM adopted a standard clear specimen shear block test method that was proposed by the USDA Forest Service, Forest Products Laboratory. Since its acceptance in 1927, the shear block test method has undergone one change in 1949, which standardized the loading rate (Rhude 1950). This ASTM D143 (1993a) shear block test consists of a 2- by 2- by 2½-in. clear wood specimen with a 2- by 2-in. failure surface (Fig. 1). (See Table 1 for SI conversion factors.) ASTM D143 shear block results, in conjunction with analytical techniques outlined in ASTM D2555 (1993d), determine small, clear shear strength values. ASTM D245 modifies the small clear shear strength values to determine design allowables for lumber. The ASTM D3737 (1993e) modifies small, clear shear strength values for glued-laminated timber. The ASTM D245 and D3737 specifications are identical except for differences in the strength ratio. ASTM D245 use a 0.75 strength reduction; whereas, ASTM D3737 uses 1.0 for the strength ratio because of the grading rules.

The ASTM D143 shear block results are not representative of the shear strength of solid sawn or glued-laminated beams. Shear block results are influenced by a stress concentration at the re-entrant corner of the test specimen. Further beam defects, such as checks, splits, and knots, are ignored in shear block testing. Shear blocks do not result in pure shear failures because moments and local stress concentrations are present. Other test methods, which create a realistic shear failure, have shown 33 percent greater shear strength (Rhude 1950).

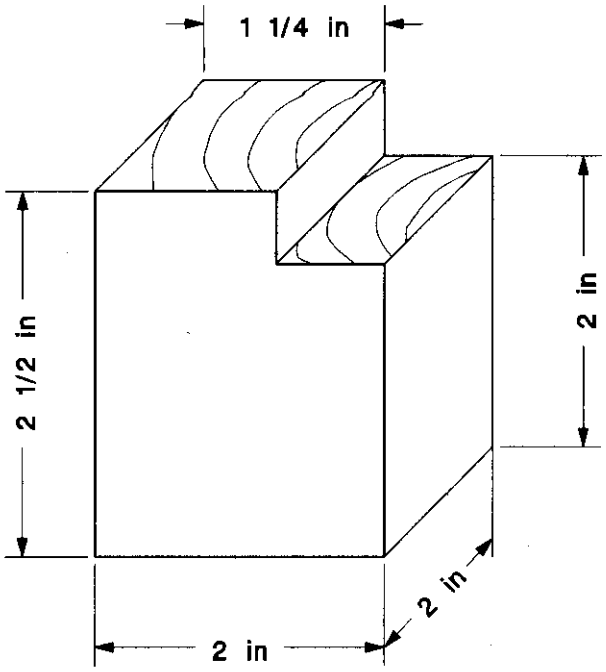


Figure 1—ASTM shear block dimensions.

Many investigators have tried to quantify the stress concentration value and stress distribution in shear blocks through experimental and analytic methods. In 1935, Coker and Coleman performed photo-elastic experiments on all known standard shear test methods. They constructed and tested a 6/10 scale xylonite model of the ASTM shear block. Xylonite is an isotropic, photo-elastic material; therefore, this material did not represent the true anisotropic nature of wood but gave insight into the stress concentration problem. Coker and Coleman arrived at two relevant conclusions from specimens loaded in the elastic regime: (1) a nonuniform shear stress distribution is present across the failure region and (2) the maximum shear stress is greatest and at least twice the average shear stress value at the reentrant comer.

Yavorsky and Cunningham (1955) investigated the strain distribution in a glued maple block by brittle coating techniques. Complete strain-field analysis of the maple blocks resulted in two general observations. First, high stress exists near the reentrant comer of the shear block. Second, the coating revealed a lack of a pure shear condition as assumed along the failure plane.

Radcliffe and Suddarth (1955) also investigated stress patterns in an ASTM shear block, a modified shear block, and a notched beam specimen. The modified shear block was the same as the ASTM standard with a horizontal slot cut into the top of the shear block to reduce the effect of the re-entrant comer. The notched beam was a rectangular beam with a slot cut at the neutral axis of the specimen. Comparisons made between each test configuration determined which method gave the most reliable value of shear strength.

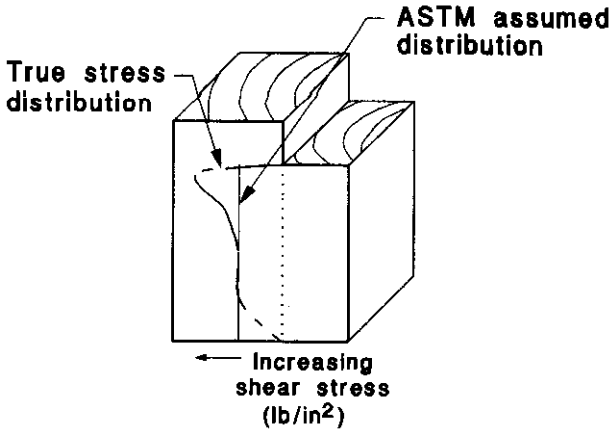


Figure 2—Comparison of assumed and true stress distribution in ASTM shear block test (Radcliffe and Suddarth 1955).

Radcliffe and Suddarth's (1955) measured and assumed stress distributions are plotted for the ASTM standard block (Fig. 2). The stress at the reentrant corner is approximately twice the assumed value. Stress distributions for their modified shear block and notched beam indicated some concentration effects, approximately 1.5 times that assumed for the modified block and approximately 1.2 times that assumed for the notched beam. Their recommendation was to use the notched beam to determine shear strength. They did not recommend the use of the modified shear block, although the values have less error, because the slot caused a high number of tension perpendicular to grain failures. Their results indicate a stress concentration factor of about 2.0 for the ASTM shear block.

Cramer and others (1984) applied a two-dimensional orthotropic fracture element to the ASTM shear block. Finite element (FE) analysis revealed a shear stress distribution similar to Radcliffe and Suddarth's (1955) experimental results, and a stress concentration factor of 2.36 near the re-entrant corner. Inability of the FE analysis to model localized crushing and splitting near the corner could account for the differences between the FE analysis and experimental stress concentration factors. Typically these effects would redistribute the stress and reduce the stress concentration.

Because ASTM shear blocks are problematic, researchers shifted their attention to investigating the shear strength in full size beams. Keenan (1974) indicated that shear strength of Douglas-fir glued-laminated beams depended on the sheared area. Also, he observed the two-beam theory proposed by Newlin and others (1934) is not applicable for glued-laminated beams. Soltis and Gerhardt (1988) reaffirmed that the two-beam theory is not applicable for glue-laminated and solid sawn lumber because of incorrectly assumed shear stress distribution.

Foschi and Barrett (1976) used Wiebull's theory of brittle fracture to assign a probability of failure based on the volume of wood under longitudinal shear. They constructed and verified several models with experimental results published by others and recommended that published allowable shear stresses be adjusted for beam size and type of loading. In 1977, Foschi and Barrett proposed a design method to account for variation in shear strength as a result of beam size and loading. This procedure applies Wiebull weak-link theory (Wiebull 1939) to a modified shear stress field. Modification accounted for the change in shear stress distribution near supports or load points. Canadian Standards Association (1984) has incorporated this procedure, as an option, for shear design.

Longworth (1977) verified experimentally that ASTM shear block strength is unrepresentative of beam strength and that beam size, sheared area, or volume appreciably affect shear strength. Quaile and Keenan (1978) introduced a beam shear test to investigate maximum shear in rectangular beams of specially designed glued-laminated material. Inner laminations consisted of the material to be investigated for shear strength. Outer laminations, with thickness equal to 0.125 the total beam depth, consisted of hickory that is high in tensile and compressive strength. This test specimen was successful in producing shear failures in 104 of the 108 specimens tested and gave further evidence that the ASTM shear block test produces lower strength values than is evident in small rectangular beams. The special test specimen was further used to determine shear strength of spruce glued-laminated beams (Keenan and others 1985). Keenan and others (1985) concluded that shear strength is not a function of volume but of sheared area. This finding conflicts with volume-based design methods proposed by Foschi and Barrett (1977).

Methods and Materials

Shear and bending strength values of matched Southem Pine and Douglas-fir glued-laminated beams of several different sizes were determined experimentally. A proposed five-point loading test was investigated to determine its effectiveness as a beam shear test when using a wide range of specimen sizes. Shear strength results from different size beams determined the variation of shear strength with beam size. Small ASTM D143 blocks cut from sheared specimens formed the data base to determine the relationship between beam strength and shear block strength. Finally. standard third-point bending specimens were tested to determine modulus of rupture (MOR). The matched shear strength and rupture values determined if a correlation exists.

Specimens

The size, beam lay-up, and numbers of Southern Pine and Douglas-fir glued-laminated specimens are presented in Figures 3, 4, and 5 and Tables 2 and 3, respectively.

Southern Pine Glued-Laminated Beams

Southern Pine source beams, 3 by 11 in. and 5 by 22 in., were obtained from a commercial glued-laminated beam fabricator. These beams were a 24F-V5 combination as specified by AITC 117-88 (1988) with written verification from the fabricator that 50 percent of the middle laminations consisted of No. 2 medium grain (MG) material. These 3- by 11-in. and 5- by 22-in. source beams were 33 and 55 ft long, respectively. End-matched specimens of proper length for shear and bending tests were cut from each source beam. These specimens were stored outside under a tarp and were tested in an uncontrolled environment.

Smaller beams, 2½ by 5½ in. and 1½ by 4 in. (approximate size), were cut from undamaged regions of the larger beams after testing. The 2½- by 5½-in. specimens consisted of two different lamination configurations depending on where they were cut from in the larger. original 5- by 22-in. beams. From the original 5- by 22-in. beam bending specimens, the outer four laminations were removed, producing smaller specimens containing high-quality laminations of the original beam, and designated HQ (High Quality). From the 5- by 22-in. beam shear specimens, the inner laminations were removed. These specimens consisted of 4, No. 2 MG laminations and designated by LQ (Low Quality). For the smaller beams, the bending and shear specimens were side-matched by cutting in half the 5-in.- wide original beams.

Only the inner laminations of the original 3- by 11-in. shear specimens were used for the 1½· by 4-in. specimens. These specimens consisted of 3, No. 2 MG laminations. Matched bending and shear specimens were obtained by cutting the original 3-in. wide members into two 1½-in.-wide side-matched specimens. Figure 3 shows the lamination lay-ups for all Southern Pine glued-laminated beams tested, and Table 2 indicates average specimen size and number of specimens tested at each size.

Douglas-fir Glued-Laminated Beams

Douglas-fir 3- by 12-in. and 5- by 24-in. source beams were obtained from a commercial glued-laminated fabricator and were either end- or side-matched. These beams were a 24F-V8 combination as specified by AITC 117-88 with written verification from American Paper Association that 50 percent of the middle laminations consisted of L2 material. These source beams were sawn into end-matched shear and bending specimens. Smaller beams, 1½ by 5 in. and 2½ by 6½ in., were cut from the undamaged regions of the tested larger beams, 3 by 12 in. and 5 by 24 in.

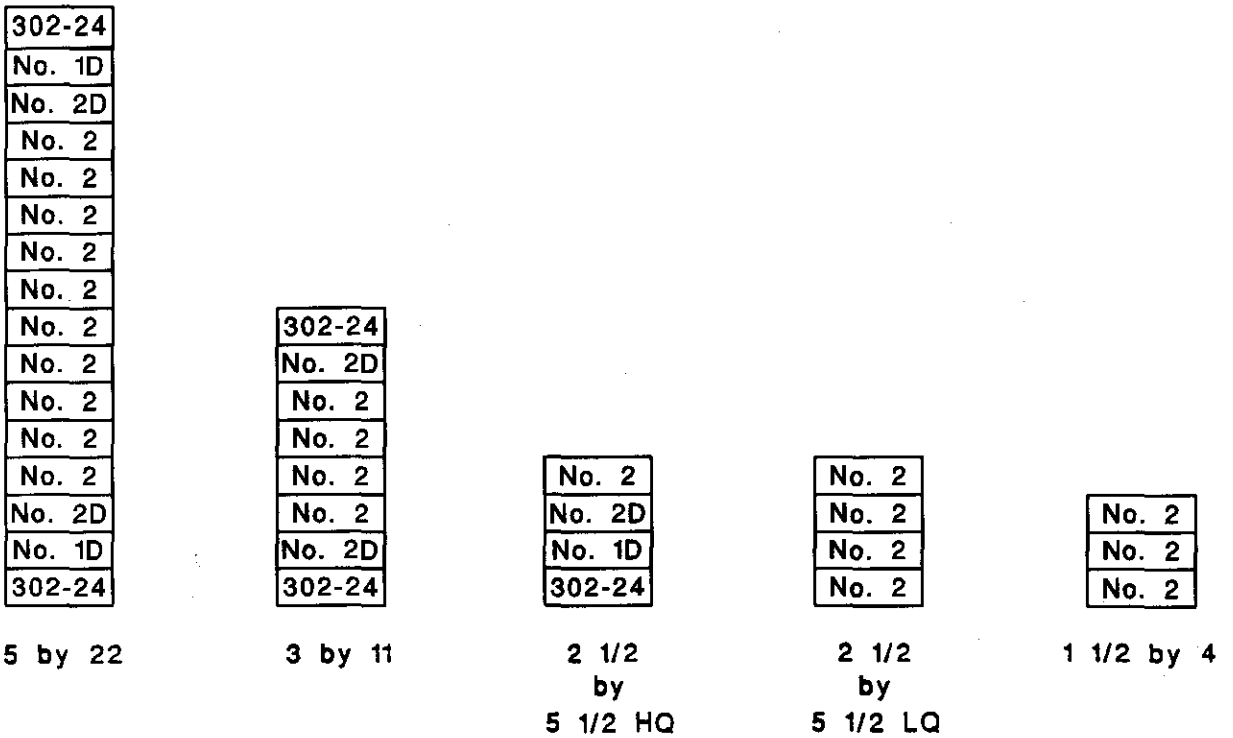


Figure 3—Southern Pine beam lamination description.

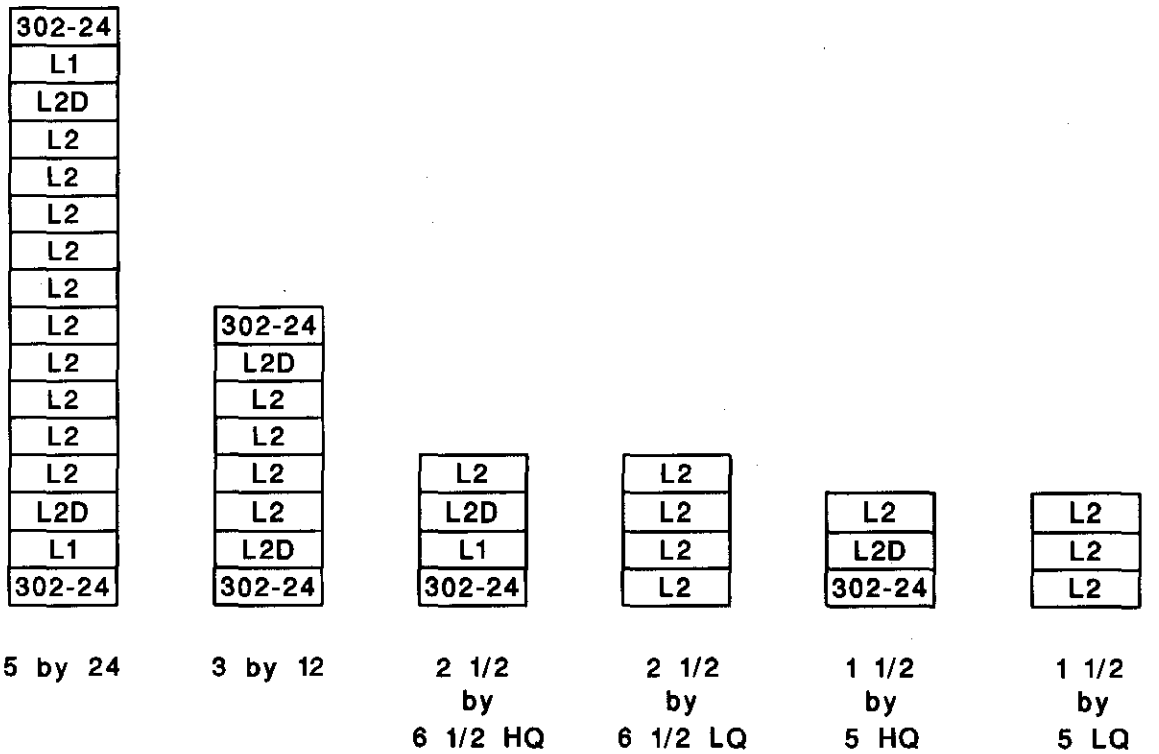


Figure 4—Douglas-fir beam lamination description.

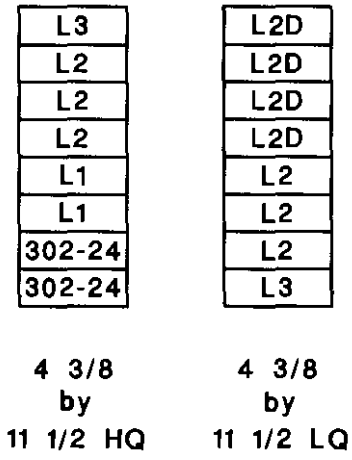


Figure 5—Washington State Douglas-fir beam lamination description.

For the 2½- by 6½-in. specimens, two types of lamination lay-ups were obtained from the original 5- by 24-in. specimens. From the original 5- by 24-in. tending specimens, the outer four laminations were removed; therefore, these smaller specimens contained high quality (HQ) laminations of the larger beam. From the 5- by 2-in. beam shear specimens, the inner laminations were removed. This resulted in specimens consisting of 4, L2 laminations and are indicated by lower quality (LQ). Inner laminations consisted of L2 material because the fabricator could not ensure that all constructed beams were at least 50 percent L3 material. To maintain consistent material in each specimen and between specimens, L2 grade material was substituted. Both bending and shear specimens were side matched by cutting the 5-in.-wide original beams in half.

For the 1½- by 5-in. size, two types of lamination lay-ups were obtained from the original 3- by 1-in. specimens. As with the 2½- by 6½-in. specimens, two grades (HQ and LQ) of 1½- by 5-in. specimens were obtained by removing either the outer or inner three laminations of the original 3- by 12-in. shear or bending specimens. Side-matched bending and shear specimens were obtained by cutting the original 3-in.-wide-members into two 1½-in. specimens. Figure 3 shows lamination lay-ups for Douglas-fir glued-laminated beams tested, and Table 3 indicates average specimen size and number of specimens tested at each size.

Final Douglas-fir specimen size was obtained from 24F-V4, 8¾-in. by 48-in. by 69-ft 0-in. glued-laminated beams originally tested in two-point bending for an unrelated study at Washington State University. Test specimens were cut from the first 16 ft of undamaged section from either end of the larger beams and ripped to consist only of the top or bottom eight laminations. The top eight laminations, which lack specially graded tension laminations, were grouped together and designated as LQ, whereas the bottom eight laminations were grouped together and designated as HQ. Side-matched specimens were approximately 4$^{3}/_{8}$ in. by 11½ in. by 16 ft 0 in. with lamination configurations shown in Figure 5.

Beam Shear Tests

A five-point bending test setup was used to maximize the number of shear failures. This setup was previously used for advanced fiber composites at NASA (Jegly and Williams 1988), for Oriented Strand Board at Purdue (Batemen and others 1990), and Australian experiments on shear strength (Leicester and Breitinger 1992). The setup consisted of a two-span beam with concentrated loads symmetrically placed on either side of the center support. This setup results in larger shear forces than could be obtained by a single-span test setup. Each setup had an overall length of 10d, where d is the nominal depth of the specimen, and individual center-to-center spans of 5d. Figure 6 shows the general setup, and Table 4 lists actual dimensions for all tests performed.

Without special attention to the load application points, the large loads needed to produce shear failures could create compression perpendicular to grain failures. To limit the compression effects, the load and support plates were designed so that at failure the perpendicular grain stresses did not exceed 1,000 lb/in.^{2} Because shear strength of the beam increased with smaller sizes, proportionally higher perpendicular compression stresses were developed for smaller beams.

Bending Tests

Third-point bending tests on simple spans were conducted to determine bending strength of Douglas-fir and Southern Pine glued-laminated beams. All tested beams had span-to-depth ratios of 15d, were d is the nominal depth of the beam. Loads were placed symmetrically around the centerline at a distance of 2.5d. Small flat-sliding plates distributed the load so that compression perpendicular to grain was not a problem. Figure 7 shows the general setup, and Table 5 lists actual dimensions for all beam tests performed.

Failure Definition

A monotonically increasing load was applied until failure in both bending and shear tests. This load was applied at a rate that conforms to ASTM D 198 (1993b), which specifies maximum load to be attained between 6 and 20 min with failure ideally occurring at 10 min.

Specimens failed in one of three modes: bending caused by tensile rupture or compressive wrinkles, shear, or compression perpendicular to grain. Some tests were halted before reaching failure because specimens rotated in the test setup to cause an unsafe condition. In this case, the maximum load before stopping the test was recorded for stress calculation. These loaded specimens were not retested.

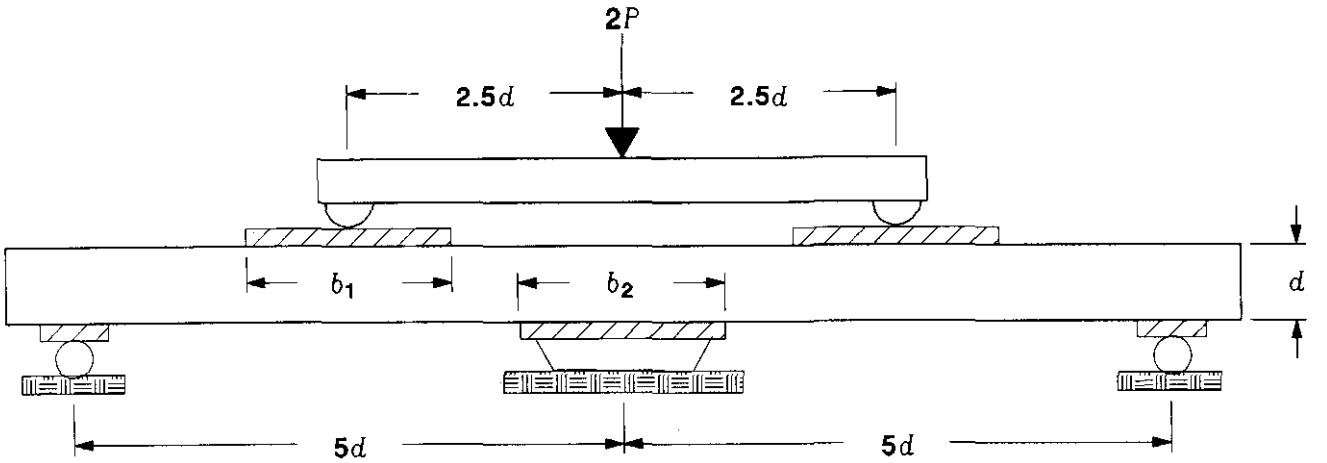


Figure 6—Five-point loaded shear test setup.

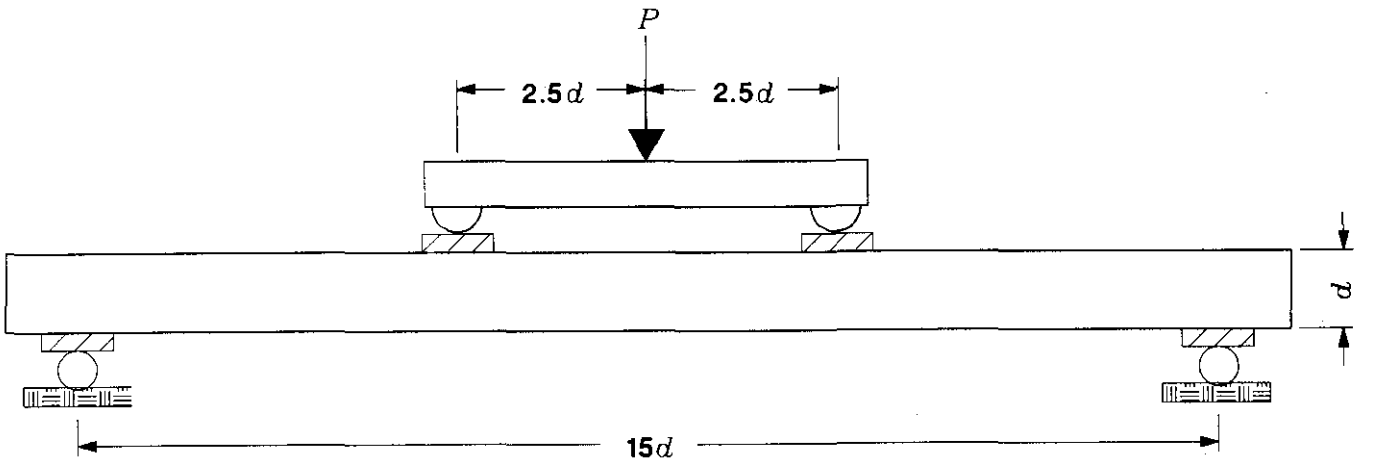


Figure 7—Third-point bending test setup.

Shear failures are not easily observed in a five-point bending test. Cracks propagate from the highly shear stressed middle half of the beam and continue until shear stress is reduced, near the load points. The crack does not propagate to the end of the beam to create a visible displacement for failure identification. Also, the rough surface texture of the wood ma& shear failure identification difficult. Vertical lines were drawn on the specimen before testing so after failure the relative longitudinal displacement of the lines could indicate the location and approximate length of the shear crack. Failure was defined by the development of a visible crack and the associated decrease in measured load.

Measurements

Measurements included specimen sizes, moisture content, maximum load at failure, and for bending specimens, the mid-span deflection Width and depth dimensions were measured to the nearest 0.01 in. at load points before testing. The size of the original beams precluded conditioning before testing; therefore, specimens were tested in an uncontrolled environment. Moisture content readings were taken after testing at varying locations within the failure zone with an electrical resistive moisture meter.

Three types of screw-driven machines with different load-measuring sensitivities were used corresponding to estimated failure loads. Only maximum load at failure was recorded for the beam shear tests, whereas loads were continuously recorded until failure for bending tests to determine modulus of elasticity (MOE). The 5- by 2-in. and 3- by 11-in. Southern Pine and the 5- by 24-in., 3- by 1-in., and 4$^3/_8$ by 11½-in. Douglas-fir specimens (large specimens) were tested with a 1,000,000-lb capacity machine. Remaining Southern Pine shear and Douglas-fir specimens were tested with a 175,000-lb capacity machine. Small-size Southern Pine bending specimens were tested with a 60,000-lb capacity machine. Load cell accuracies and loading rates for all tests are listed in Table 6.

For the bending specimens, deflection measurements were recorded by two methods. The first method, used for the large specimens, was to read a ruler that measured deflection from a wire stretched between two nails located at the neutral axis at each end of the beam. Deflections were read at

regular load intervals to the nearest 0.01 in. The second method, used for the smaller specimens, involved attaching a linear variable differential transducer (LVDT) at mid-span to continuously record deflections with a computer data acquisition system.

Orientation of Glued-Laminated Beams in Test Setup

Typically, glued-laminated beams have high quality outer laminations to resist tensile tending stresses. Tension laminations am placed to resist the maximum moment produced from loading. For bending tests, the lamination was placed at the bottom to resist maximum tensile stresses caused by a positive moment. For beam shear tests, the lamination was placed at the bottom to resist moment under the load points. During preliminary five-point tests, most bending failures occurred under the load point instead of over the middle support. The size of the middle plate most likely reduced the moment over the center support.

Shear Block Tests

From each beam that failed by shear, an ASTM shear block specimen was cut from an undamaged section. Southern Pine shear blocks were cut from two locations. Shear block specimens for the 5- by 22-in. and 3- by 1-in. specimens were cut from the beam end at the same elevation as the shear crack. For the remaining Southern Pine specimens, shear blocks were cut from the same lamination as the beam shear crack and in a region absent of the crack. All Douglas-fir shear blocks were obtained from the second location described for the Southern Pine shear blocks (within the same lamination). All blocks were cut with the same grain orientation as in the original beam.

Testing procedures conformed to ASTM D 143, except that the grain orientation was not at 0° or 90°. Before testing, shear blocks were conditioned to about 12 percent moisture content. After testing, moisture content and specific gravity were determined from damaged shear block specimens.

Results

Experimental result averages and coefficients of variation for beam shear, flexural strength, and shear blocks are summarized in Tables 7–12. Individual Southern Pine and Douglas-fir glued-laminated data are presented in Appendix A and B, respectively.

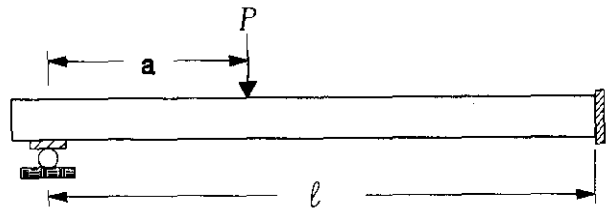


Figure 8—Five-point loaded shear test idealization.

Maximum Stress Calculations

Five-Point Bending

Shear strength values were determined by idealizing the 2-span beam as a single-span beam with one end fixed and the other end hinged with the span length equal to the center-to-center support distance, as shown in Figure 8. Cowan (1962) experimentally showed that near loading plates the shear stress distribution changes resulting in a maximum 33 percent increase in shear stress and with the maximum stress occurring near the top or bottom of the beam. Cowan also noted this redistribution was within a distanced from the end of the loading plates. Liu and Cheng (1979) analytically modeled this variation using orthotropic beam theory. We observed shear cracks occurring, on average, within the middle quarter of the beam. Therefore. a parabolic shear stress distribution was assumed. This assumption allowed calculation of shear stress techniques consistent with current National Design Specifications (AFPA 1992).

The equation for shear in a beam fixed at one end and supported at the other under a concentrated load is given by

$$V = \frac{Pa}{2\ell^3}(3\ell^2 - a^2) \qquad (1)$$

where

V = shear force (lb)

P = load applied to span (lb)

ℓ = span length (in.)

a = distance from load to roller support (in.)

The load measured is the sum of the load applied at mid-span of each span of length 5d. The shear force is

$$V = \frac{11P}{16} \qquad (2)$$

where

P = load applied to a single span, half of total measured load (lb)

For the assumed stress distribution. the longitudinal shear stress equation for a rectangular section is

$$\tau = \frac{VQ}{It} = \frac{3V}{2bd} \tag{3}$$

where

t = shear stress (lb/in^2)

V = shear force (lb)

b = width of beam (in.)

d = depth of beam (in.)

Q = statical moment of the area (in^3)

Substituting Equation (2) into Equation (3), the general equation that relates failure load to shear strength is

$$\tau = \frac{33P}{32bd} \tag{4}$$

Third-Point Bending

Modulus of rupture values were calculated using beam theory considering only the load applied to the specimen and neglecting the dead weight of the beam.

Flexural stresses are

$$\sigma = \frac{Mc}{I} \tag{5}$$

where

s = flexural stress (lb/in^2)

M = Applied Moment (lb-in.)

c = distance from the neutral axis of the cross section (in.)

I = moment of inertia about the neutral axis (in^4)

From Equation (5), the applied moment at failure is half the maximum load multiplied by one-third the span length. The moment of inertia for a rectangular cross section and the maximum distance from the neutral axis are substituted into the equation to give maximum flexural stress values. The maximum stress at failure or MOR is

$$\sigma = \frac{P\ell}{bd^2} \tag{6}$$

where

P = total applied load to test setup (lb)

l = span length (in.)

Equations (4) and (6) were used to calculate the shear strength and MOR.

Southern Pine Glued-Laminated Beams

Beam Shear Tests

Table 7 lists the moisture content, average shear strength (determined by Equation (4)), shear stress at failure, and coefficient of variation for each beam size tested. The average shear strength was calculated using only the results of the beams that failed in a shearing mode. Shear stress at failure was calculated using the maximum recorded load regardless of the failure mode. The shear strength calculated by either including or excluding failures other than shear failures gives approximately equal results and a coefficient of variation about 10 percent. Cumulative distributions of shear strength for each beam size, including beams that failed io modes other than shear, are also shown in Figure 9.

Shear failures were observed in 102 of 138 tests under five-point loading. A typical shear failure is shown in Figure 10. The 74 percent shear failure rate may have been greater if all tested material was specifically manufactured to have a high quality lamination to resist flexural stresses. Specimens cut from previously tested material, mainly specimens with the LQ designation, did not have specially graded tension laminations to resist the high flexural stress.

Bending Tests

Modulus of rupture was calculated using Equation (6), which assumes elastic response until failure. All 138 beams tested failed either in flexural tension on the bottom side of the beam or flexural compression parallel to the grain on the top side of the beam. Only a few compression failures were observed in the high quality 2½- by 5½-in. and 1½- by 4-in. specimens. Tables in Appendix A indicate the type of beam failure for each specimen. Table 8 Lists average MOR, coefficient of variation, and average moisture content for each beam size tested. Distributions of MOR for each beam size are shown in Figure 11. In most distributions, experimental results are truncated at the upper end of the distribution, indicating finger joint failures rather than wood failures (Appendix Tables A.1 and A.2).

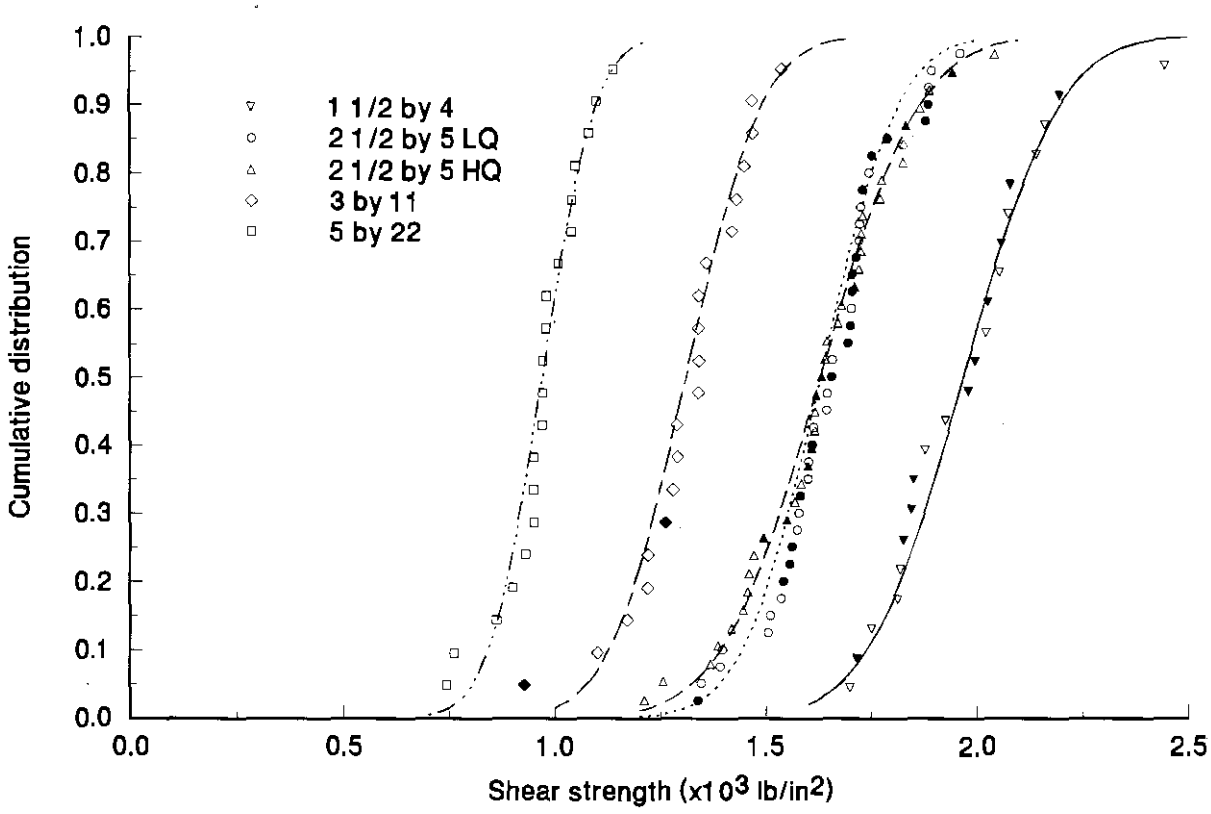


Figure 9.—Cumulative distribution of Southern Pine beam shear strength at all tested beam sizes. Solid symbols indicate shear strength attained during a nonshear failure of the beam. The line represents normal distribution based on the experimental mean and standard deviation of shear specimens of that size. In most distributions, experimental results approximate a normal distribution.

Shear Block Tests

A shear block specimen was cut from each glued-laminated beam shear specimen that failed by shear. The specific gravity, moisture content, and shear strength were determined for each shear block. Average values and coefficients of variation for shear strengths are listed in Table 9, and the cumulative distribution of shear block data is shown in Figure 12. The shear block values and variations were representative of average published values: 1,390 lb/in^2 to 1,680 lb/in^2 (Forest Products Laboratory 1987) for the dry Southern Pine, loblolly, longleaf, shortleaf, and slash pine. Specific gravity results were also in the typical range for Southern Pine, 0.51–0.59. More specifically, the shear strength and specific gravity values are indicative of only the shortleaf and loblolly species of Southern Pine.

Douglas-Fir Glued-Laminated Beams

Beam Shear Tests

Table 10 lists the moisture content, average shear strength, shear stress at failure for all tests, and coefficient of variation for each beam size tested. Again, shear strength and shear stress at failure are approximately equal. Also, an 8 percent coefficient of variation for Douglas-fir shear strength is about the same as Southern Pine shear strength. Cumulative distributions of shear strength for each beam size, including specimens that failed in modes other than shear, are shown in Figure 13. For most distributions, experimental results approximate a normal distribution.

Shear failures were observed in 170 of 192 tests under five-point loading (89 percent). As with Southern Pine, the Douglas-fir specimens cut from the inner lamination of the original beam had a higher percentage of bending failures because of the absence of the tensile lamination.

Bending Tests

Most of the 192 beams tested failed in bending either in tension or compression parallel to the grain on the top of the beam. Some beams failed by propagation of an existing crack in the specimen. Tables in Appendix B indicate the type of beam failure for each specimen. Table 11 lists average MOR, coefficient of variation, and average moisture content for each beam size tested. Distributions of MOR for each beam size are shown in Figure 14. In most distributions, experimental results are truncated on the upper end of the distribution indicating strength reducing flaws (finger joints or knots) (Appendix Table B.1 and B.4).

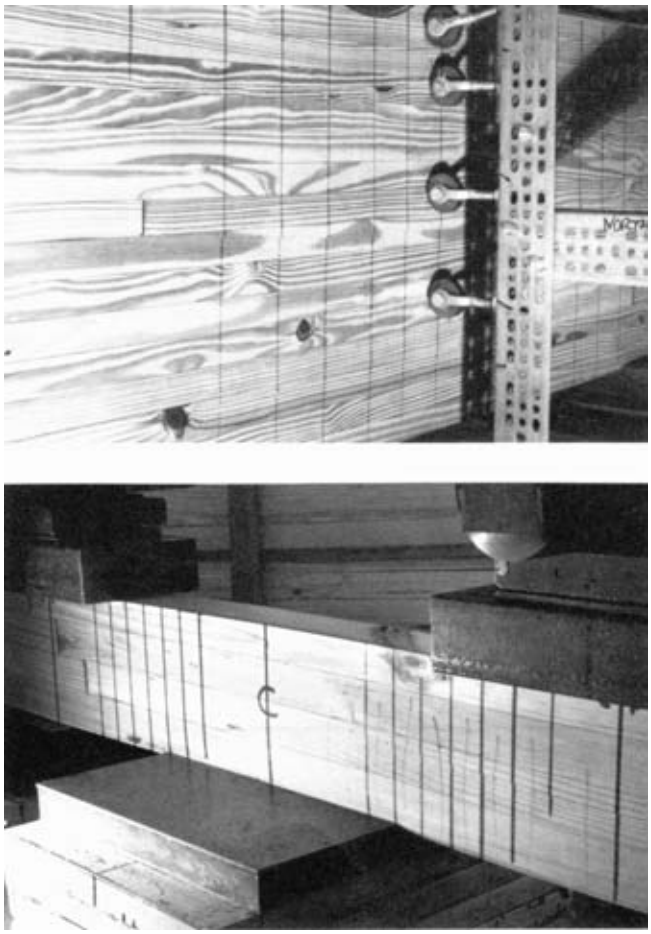


Figure 10.—Typical shear failure in a 5 - by 22 -in. Southern Pine beam (M910285-3) (top) and a 2½- by 6½-in. Douglas-fir beam (M910282-2) (bottom).

Shear Block Tests

A shear block specimen was cut from each glued-laminated beam shear specimen that failed by shear. The specific gravity, moisture content, and shear strength were determined for each shear block. Average values and coefficients of variation are listed in Table 12. A cumulative distribution of shear block data is shown in Figure 15. The shear block values are representative of published average strength and variability values (Forest Products Laboratory 1987) for Douglas-fir, 1,130 lb/in^2 to 1,510 lb/in.^{2} Most specific gravity results are within the typical range for Douglas-fir, 0.46 to 0.48. The shear blocks taken from the 2½- by 6½- in. HQ and 5- by 24-in. beams had a lower specific gravity and larger shear strength coefficient of variation. Additionally, the shear blocks taken from the 5- by 24-in. beams had a lower shear strength than did the other samples. We think these unexpected specific gravity, shear strength, and coefficient of variation results were from a large number of juvenile wood samples included in these data sets.

We observed more than 50 percent of the 2½- by 6½-in. HQ; 80 percent of the 5- by 24-in. shear block samples had less than six growth rings per inch. We observed 15 percent of the 2½- by 6½-in. HQ; 33 percent of the 5- by 24-in. shear blocks samples had less than four growth rings per inch.

Discussion

Observed Results

The five-point bending test procedure produced 272 shear failures out of 330 tests (82 percent). In general, these failures initiated in the wood of the highly stressed region between the load supports and rarely propagated past the loading points. Cracks typically followed the grain between the early and late wood. At crack initiation, a substantial load reduction was always noted. In some cases, a sustained load after shear failure produced a bending failure. This is reasonable because after failing in shear the cross sectional moment of inertia is reduced by 2/8. Therefore, if the load reduction after cracking was not significant, the remaining cross section could fail in bending. Overall, use of a five-point loading configuration for evaluation of shear strength of unchecked glued-laminated beams is a reliable method producing a high percentage of shear failures. It is particularly useful when testing smaller sized beams which are difficult to fail in shear. It also is representative of actual continuous beam conditions in which shear stresses problems might arise.

Of the 58 specimens that did not fail in shear, 33 failed in bending, 11 rotated in the testing frame, 12 had excessive compression perpendicular to grain under loading points, and 2 exceeded the load cell capacity. Lateral supports and slightly larger loading plates should be used to reduce the number of nonshear failures (observed primarily in the smaller specimens).

Overall, the shear and bending strength variations are consistent with accepted values. Coefficients of variation for MOR occur within or close to the accepted range of 15 to 20 percent. The ASTM shear block coefficients were comparable to the 14 percent previously published (Forest Products Laboratory 1987). Beam shear coefficient of variation is less than both the ASTM shear block variation and the bending values. Lower variation than bending is attributed to insignificant effects of knots and finger joints on shear strength which are included in bending COVs. Lower COV for beam shear than ASTM shear block is attributed to the ability of the crack to develop along a plane of weakness in the beam. In ASTM shear block, the failure is forced to occur over a specific region regardless of material strength variability.

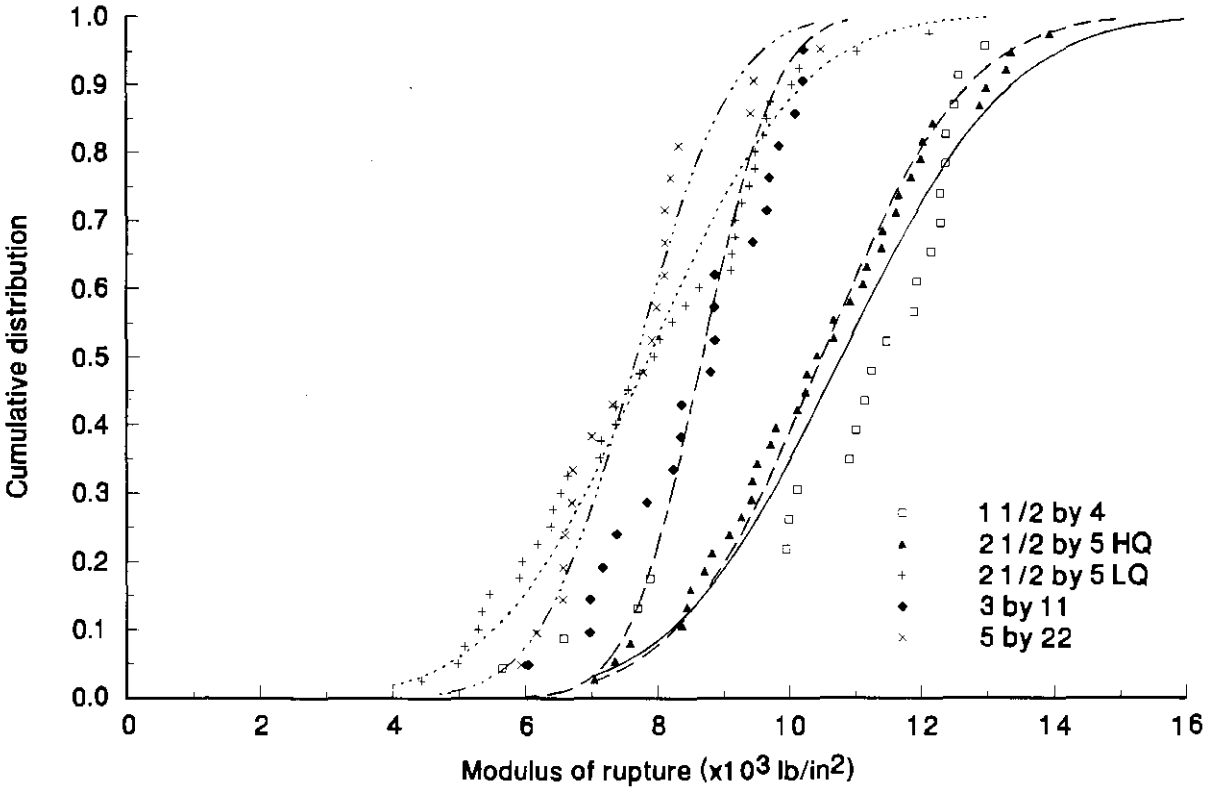


Figure 11.—Cumulative distribution of Southern Pine MOR at all tested beam sizes. Line represents normal distribution calculated based on the experimental mean and standard deviation.

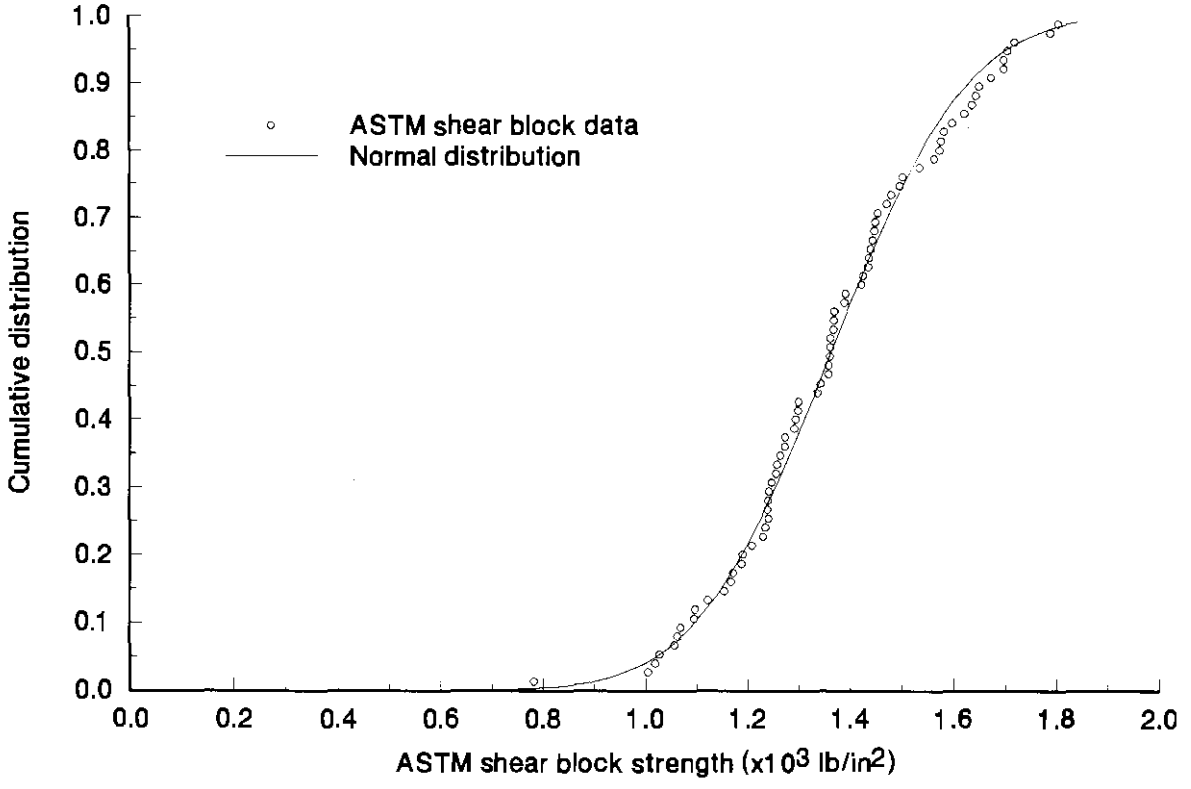
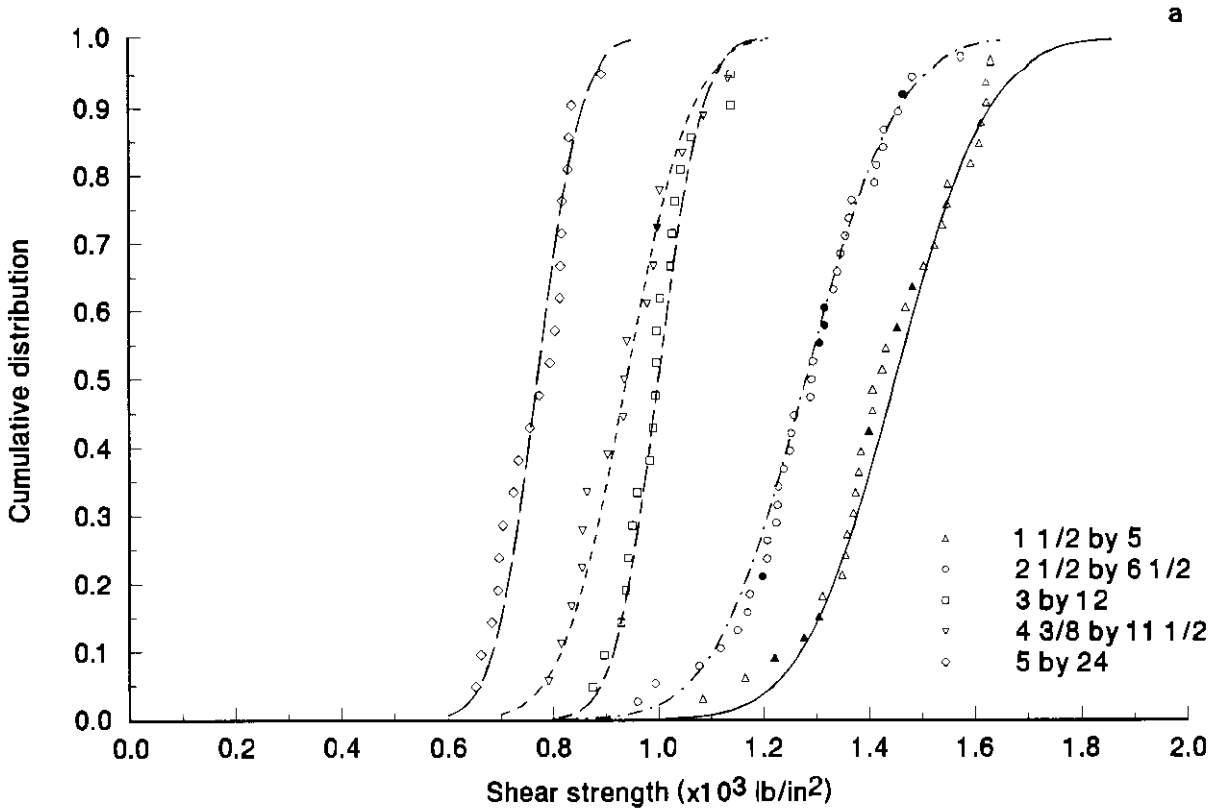


Figure 12.—Cumulative distribution of Southern Pine ASTM shear block strength



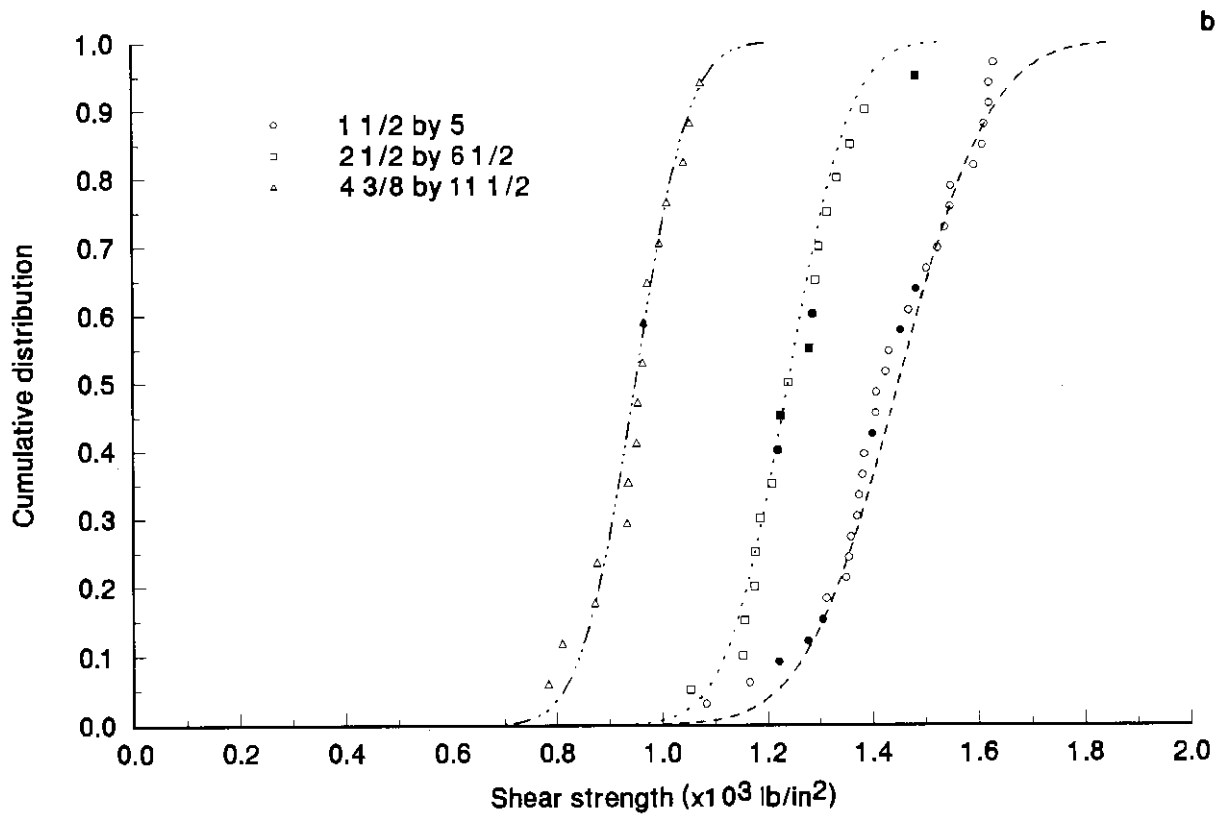


Figure 13.—Cumulative distribution of Douglas-fir beam shear strength at all tested beam sizes. (a) Source and HQ beams (b) LQ beams. Solid symbols indicate shear strength attained during a nonshear failure of the beam. The line represents normal distribution based on the experimental mean and standard deviation of shear specimens of that size.

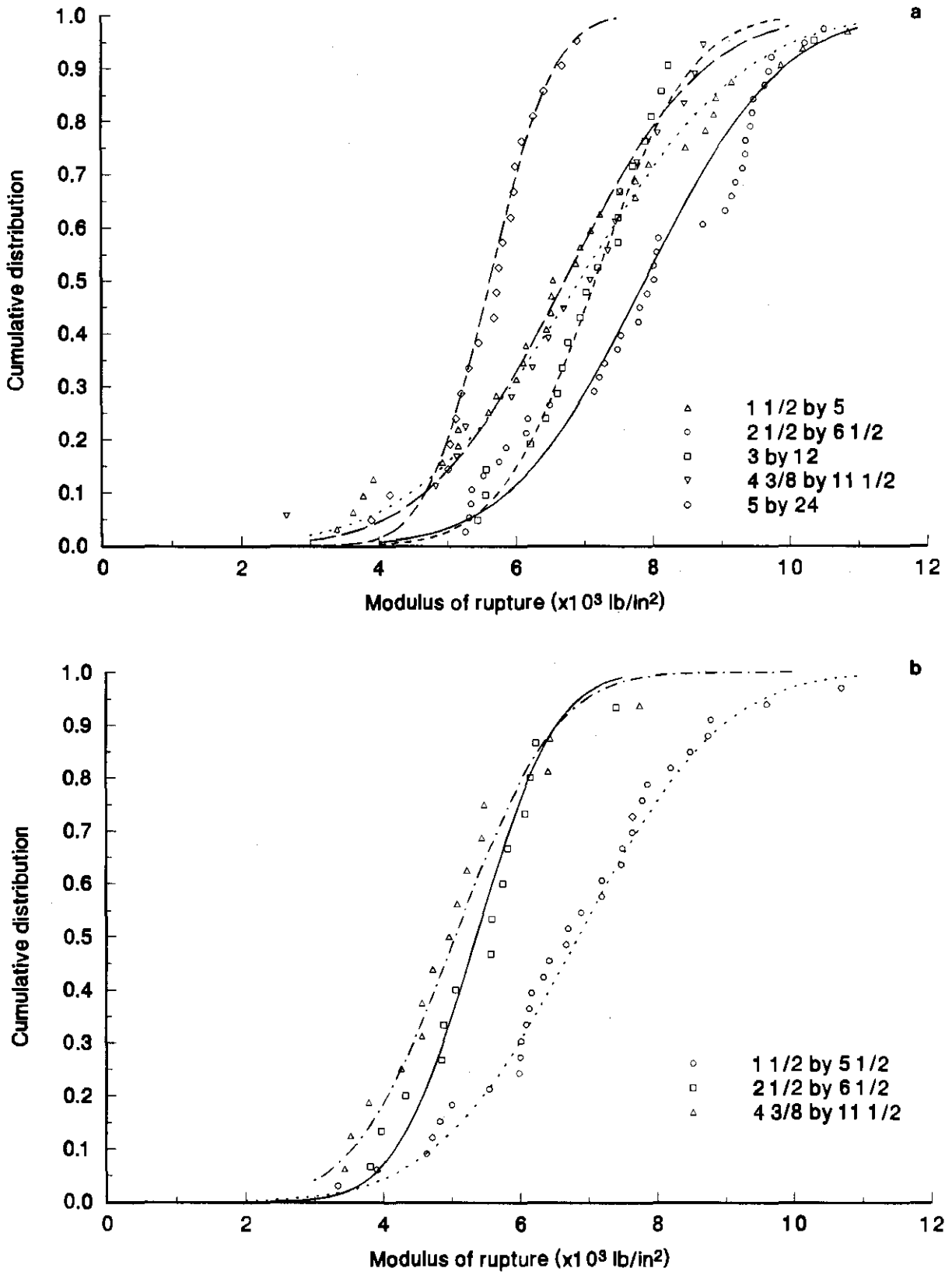


Figure 14—Cumulative distribution of Douglas-fir MOR at all tested beam sizes. (a) Source and HQ beams (b) LQ beams. The line represents the normal distribution calculated based on the experimental mean and standard deviation.

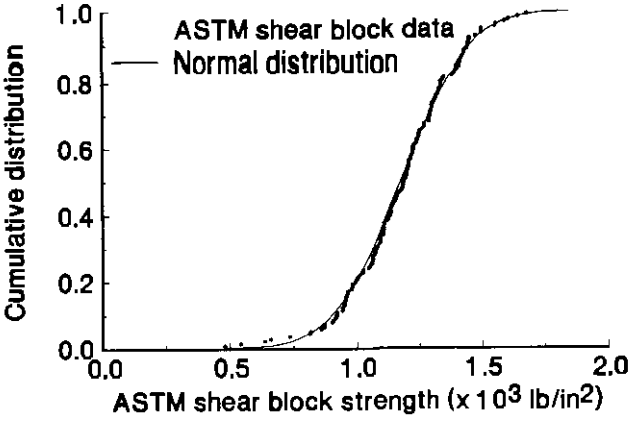


Figure 15—Cumulative distribution of Douglas-fir ASTM shear block strength. The line represents normal distribution.

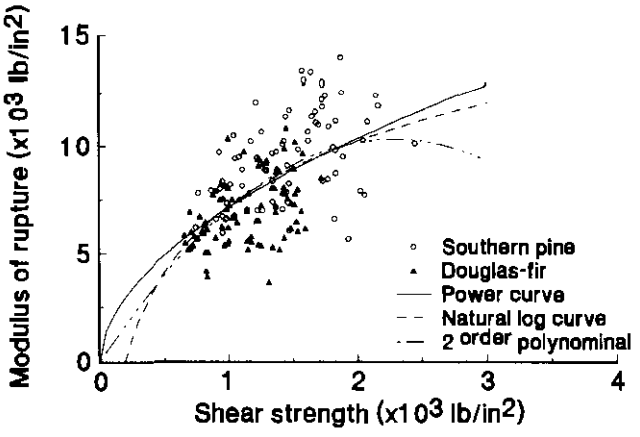


Figure 16.—Modulus of rupture and shear strength for Southem Pine and Douglas-fir.

Shear/Bending Correlation

Shear strength compared to MOR is presented indicating some results of a regression analysis to see if a correlation exists for Southern Pine and Douglas-fir glued-laminated beams (Fig. 16). Several curves were regressed through various combinations of the shear and bending strength data to uncover any relationships. Combination of data sets included the entire and censored Southern Pine results, the entire and censored Douglas-fir results, and the entire and censored strengths for the combined Southern Pine and Douglas-fir data. Censored data included only the matched pairs that had the specially graded tension laminations. Coefficient of determination (r^2) results of the regression analysis for all curves and combined sets are listed in Table 13. In general, the censored data set resulted in better determination coefficients, and so did the combined Southern Pine and Douglas-fir data set. Overall, the low r^2 values indicate no correlation between shear strength and MOR.

Further, a regressed curve should ideally pass through the origin (t = 0, MOR = 0). In structural applications, the change in moment from one point to another is equal to the shear between the two points. Therefore, shear must be present to cause moment. Using this reasoning, the log curve in Figure 16 is considered nonrealistic since it does not pass through the origin. Also, the two-order polynomial should be used with caution because it only describes the response within the observed data.

Shear/Beam Size Correlation

The effect of the stress concentration at the re-entrant corner was described previously in the background section of this paper. In summary, the ASTM assumed shear strength (τ_{ASTM}) was less than the true failure stress (τ_{fail}) by a factor of about 2.0 (Radcliffe and Suddarth 1955). The FE analysis indicates a factor larger than 2.0, but these methods cannot model effects (crushing and splitting) which might alleviate the stress concentration effect (Cramer and others 1984). Therefore, we conclude, from the available information, an appropriate estimate of the stress concentration factor for the ASTM shear block is 2.

ASTM shear strength data were adjusted to hue stress at failure by

$$\tau_{fail} = C_f \tau_{ASTM} \qquad (7)$$

where

C_f = stress Concentration factor to adjust the ASTM shear block to true stress distribution. Assumed to be 2.0.

τ_{ASTM} = ASTMD143 published shear block values

Adjusted ASTM shear stress, τ_{fail}, are assumed to occur on a 4 in² failure plane.

An analysis of variance (ANOVA) was performed on the Southern Pine and Douglas-fir shear strength data to determine if strength results for various size beams were statistically different. The ANOVA calculations were performed by a statistics software package (SAS) using a general linear model and Tukey's studentized range test for multiple hypothesis comparison at 0.05 level of confidence. Tukey Groupings indicate those sizes that are statistically different from other sizes. The results of the shear strength ANOVA are listed in Table 14; note that in the table statistically different groups are assigned different letters. Thus, for Southem Pine there is no statistical difference in mean shear strength of the 2½- by 5½-in. LQ and HQ beams. There is a statistical difference in mean shear strength between all sizes of Southern Pine beams. For Douglas-fir there is no statistical difference in mean shear strength for

either the 1½- by 5-in. and 2½- by 6-½ -in. LQ and HQ or the three largest beam sizes. There is however a difference between the three largest sizes. the three smaller sizes, and the shear block. For both species, ANOVA revealed a change in shear strength with beam size.

Next, a regression analysis was performed by regressing either beam depth, shear area, or beam volume versus shear strength. Results are given in Table 15 for several forms of linear or nonlinear regression equations. Beam depth, d, was taken as the average depth of the beams for a given beam size or beam type. Shear area was taken as the average width, b, of the beam times total length of the beam under a shear force action in one span. For the test setup used, the shear area is 5bd. For a beam with third-point loading, the shear area is 0.67 lb, where l is the span length. Our definition of shear area differs from that of Keenan and others (1985) who defined shear area as shear span times beam width, where shear span was defined as the length of the beam under positive shear. We defined shear area as the length under both positive and negative shears. This definition would result in shear area equivalent to 2x Keenan's defined area for most cases encountered in engineering design. This definition of shear area is easier to apply since in most loading cases the shear span equals the length of the beam. For a uniformly loaded beam, the shear span is conservatively taken as the length of the beam. Finally, beam volume was defined as the average width multiplied by average depth multiplied by the length of span.

The regression analysis indicates that the shear area is somewhat better in characterizing the variation of shear strength. There is equal confidence that beam volume or depth might characterize the variation in shear strength because of the slight variation in beam width. Both the logarithmic and power curve equations modeled shear strength variation well. The forms of these equations are consistent with Longworth's (1977) findings on shear strength. Longworth tested 150 glued-laminated Douglas-fir beams under four-point bending. Fabrication of the laminated beam was under the Canadian CSA 0177 Qualification (CSA 1965) for manufactured lumber. Figure 17 indicates Longworth's shear strength compared to this study using shear area as a basis. A visual comparison indicates good agreement between both data sets. A power curve is recommended to classify the variation of shear strength because of the similarities between Longworth and this study, and the higher coefficient of determination from regression analysis for the power curve.

Shear area is plotted versus shear strength in Figure 18 for Southern Pine and Douglas-fir beam specimens. For both species, shear strength decreases as shear area increases. The change decreases quickly for small shear areas and decreases asymptotically for larger shear areas. The asymptote results in small changes in shear strength for larger size beams.

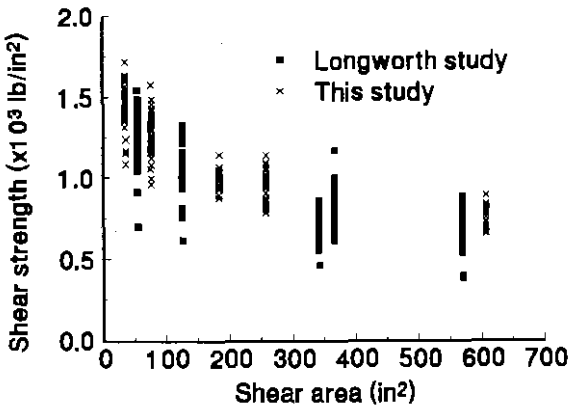


Figure 17—Comparison of Douglas-fir data with Longworth's data.

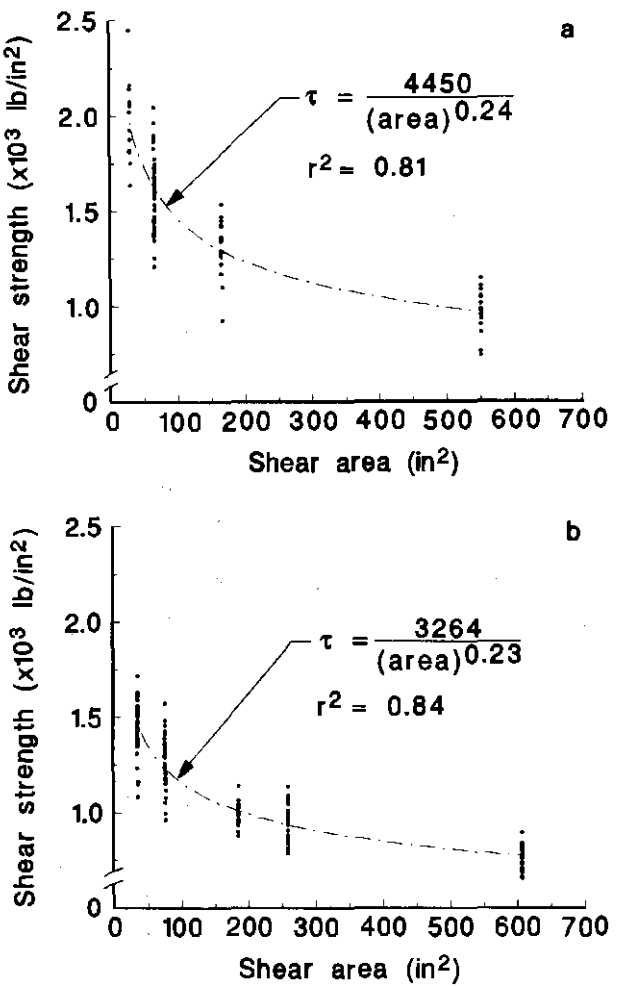


Figure 18—Beam shear strength and shear area regression for (a) Southern fine and (b) Douglas-fir.

This explains why no statistical difference was noted in the three largest sized Douglas-fir beams.

Shear strength of the Southern Pine beams tested was consistently higher than the Douglas-fir beams. This difference has also been noted in published shear block and allowable shear stress values.

Power curves were regressed independently through the Southern Pine and Douglas-fir shear strength values. The regression equations for Southern Pine glued-laminated beams are

$$\tau = \frac{4450}{A_s^{0.24}} \tag{8}$$

where τ is the beam shear strength, A_s is the shear area. and the regression coefficient $r^2 = 0.81$. The regression equation for Douglas-fir glued-laminated timber is

$$\tau = \frac{3264}{A_s^{0.23}} \tag{9}$$

The regression coefficient is $r^2 = 0.84$. Both Equations (8) and (9) have approximately equal exponents, indicating that the shape of the curve is independent of species. Also, both curves have regression coefficients, indicating a high degree of correlation between the shear area parameter and maximum shear strength. The constant in the numerator of the equation is different which reflects the difference in shear strength of the species.

Different numbers of specimens failed in shear at each beam size resulting in unequal sample size for statistical analysis. To see if unequal sample sizes affect regressions, a plot of the average shear test strengths and predicted strength was generated. No practical difference was observed between predicted strengths and average tested strength (Fig. 19); therefore, the weighting effect of the varying number of specimens at each beam size had no practical significance on the regression curves.

Shear Block/Beam Shear Relationships

Adjusted ASTM shear stress, τ_{fail}, on a shear area of 4 in^2 was plotted with the beam shear data (Fig. 20). A power curve regression, consistent with the finding of the previous beam shear regression analysis, was performed on this combined data set. The regression equation with the inclusion of shear block data for Southern Pine is

$$\tau = \frac{3660}{A^{0.20}} \tag{10}$$

with a regression coefficient of $r^2 = 0.87$.

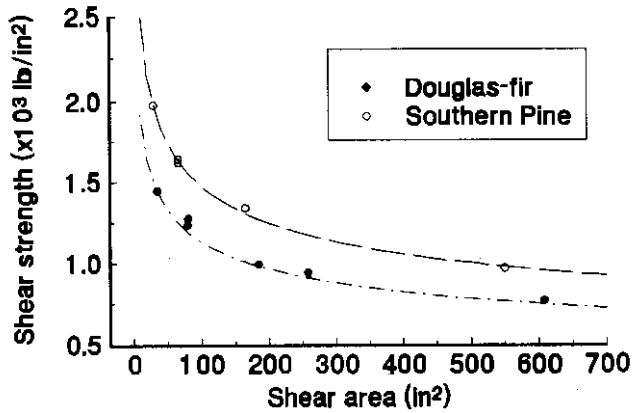


Figure 19—Comparison of average shear strength at various beam sizes with the regression line prediction.

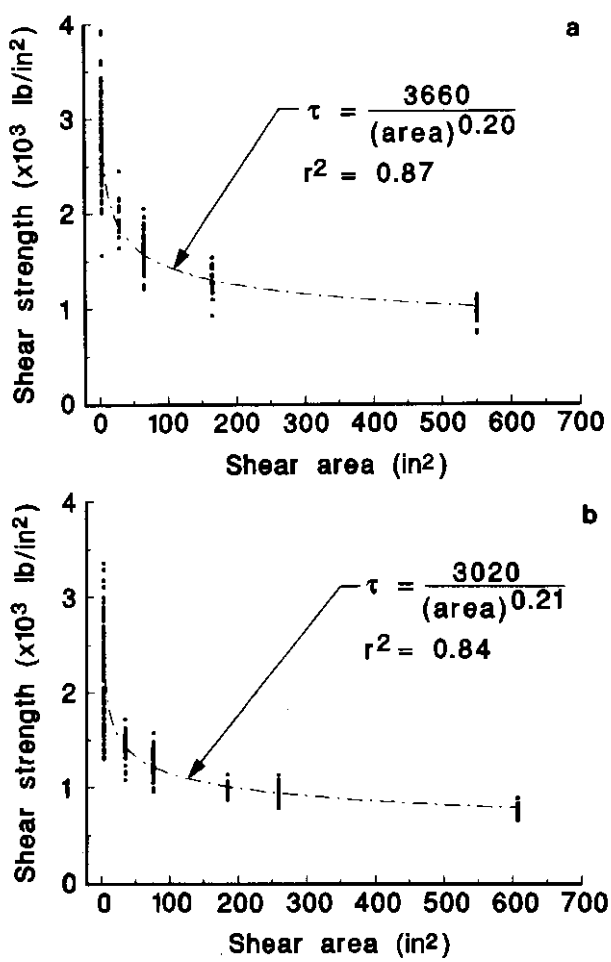


Figure 20—Shear strength and shear area regression for (a) Douglas-fir beams and block shear specimens and (b) Southern Pine beams and block shear specimens.

For Douglas-fir data, the regression equation is

$$\tau = \frac{3020}{A^{0.21}} \tag{11}$$

with a regression coefficient of $r^2 = 0.87$. The shear block data set was larger than that for the beam shear data which weights the curve. Comparison of the regression curve for the overall data with the mean values at each size is shown in Figure 21. On the basis of mean values at each beam size the weighting effect of the larger shear block data set is negligible. Regression Equations (10) and (11) for two species have approximately equal exponents. The shape of the curve is independent of species and the exponent may be taken, without any loss of generally, as 1/5. Also, both equations have regression coefficients which indicate a high degree of correlation between the shear area parameter and maximum shear strength. Again, the only difference is the constant in the numerator of the equations which reflect the difference in shear strength of the species as indicated by different ASTM shear block values. Therefore, the shear stress variation curve for different species is related to published ASTM shear block values.

The constants in Equations (10) and (11) represent shear strengths corresponding to a shear area of 1 in.^{2} The shear area of the ASTM block is 4 in.^{2} Thus, these equations are re-written in terms of ASTM shear block strength rather than the strength corresponding to 1 in^2 shear area by including an adjustment factor of $(4^{1}/_{5})/(1^{1}/_{5}) = 1.3$. The exponents in the Equations (10) and (11) are approximately equal to 1/5. Thus, we recommend

$$\tau = \frac{1.3 C_f \tau_{ASTM}}{A^{1/5}} \qquad (12)$$

where

τ = beam shear strength, ps
$C_f = 2$ = stress concentration factor to adjust the ASTM shear block strength to the true strength at failure.
τ_{ASTM} = ASTM D143 published shear block values
A = shear area = area of beam subjected to shear forces.

This recommended Equation (12) relates beam shear strength to ASTM block shear strength and depends on the shear block stress concentration factor and the beam shear area. This equation is plotted for the Douglas-fir and Southern Pine data (Fig 22). From the figures. it is seen that the recommended equation approximates the data quite well.

Care must be exercised so as not to extrapolate this equation beyond the sizes tested. There was no statistical difference in the three largest sizes of Douglas-fir beams tested indicating there is a minimum beam shear strength regardless of how large the beam is. Extrapolation to larger size beams using our recommended Equation (12) will indicate decreasing beam shear strength which is erroneous.

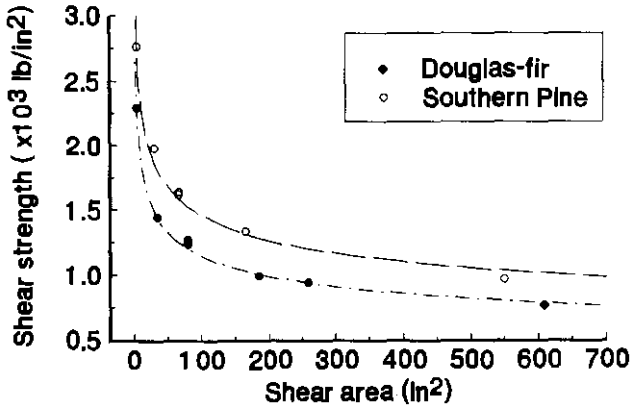


Figure 21—Comparison of average shear strength at various beam sizes and the regression line prediction.

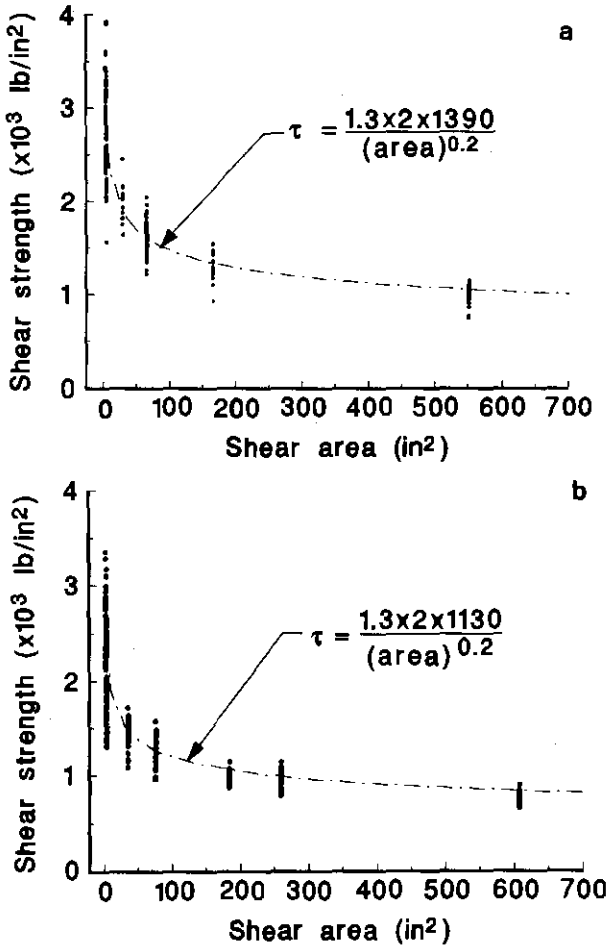


Figure 22—Recummended equation and (a) Douglas-fir and (b) Southern Pine data.

Conclusions

In summary, 331 third-point bending and two-span beam shear tests were conducted on glued-laminated Douglas-fir and Southern Fine beams. Based on these experiments, we conclude the following:

1. The two-span beam test procedure is recommended as a standard to measure beam shear. It is particularly useful when testing smaller sized beams that are difficult to fail in shear. Consistent shear failures in an unchecked beam occurred between the loading point and middle support of the beam. The shear crack did not propagate to the end of the beam.

2. A data set of both beam shear and ASTM block shear strength for Southern Pine and Douglas-fir glued-laminated beams for a range of beam sizes from 1½ by 4 in. to 5- by 24-in. cross sections was developed.

3. No correlation exists between shear strength and MOR in matched glued-laminated beams.

4. Shear strength of glued-laminated beams is not constant but varies with the sue of the beam. Larger beams have lower shear strength. Shear strength variation for different species and various size beams can be modeled with the following equation, which is based on a relationship between beam shear and ASTM shear block strength including a stress concentration factor for the re-entrant corner of the shear block

$$\tau = \frac{1.3 C_f \tau_{ASTM}}{A^{1/5}}$$

   Care must be exercised so as not to use this equation to extrapolate to beam sizes much larger than those tested.

5. From a regression analysis, the shear area parameter is somewhat better than beam volume or depth in modeling variation in shear strength.

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Table 1—SI conversion factors

English unit	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
pound-force (lbf)	4.448	newton (N)
pound-force per square inch (lbf/in^2)	6.895	Pascal (Pa)

Table 2—Southern Pine beam sizes and number of specimens tested

Beam size[a] (in.)	Actual average beam size Width (in.)	Actual average beam size Depth (in.)	Number of tested specimens
1½, by 4	1.37	4.13	22
2½ by 5½, LQ	2.38	5.64	39
2½ by 5½ HQ	2.36	5.39	38
3 by 11	3.00	12.00	20
5 by 22	5.00	22.00	20

[a] LQ is No. 2 MG inner laminations: HQ is specially graded tension laminations.

Table 3—Douglas-fir beam sizes and number of specimens tested

Beam size[a]	Actual average beam size Width (in.)	Actual average beam size Depth (in.)	Number of tested specimens
1½ by 5½ LQ	1.38	5.05	32
1½ by 5½ HQ	1.38	5.20	32
2½ by 6½ LQ	2.46	6.50	39
2½ by 6½ HQ	2.46	6.17	39
3$^1/_8$ by 12	3.07	11.91	20
4$^3/_8$ by 11½ LQ[b]	4.33	11.61	16
4$^3/_8$ by 11½ HQ[b]	4.34	11.79	17
5$^1/_8$ by 24	5.06	23.85	20

[a] LQ in L2 inner laminations; HQ is specially graded tension laminations.
[b] Originally tested at Washington State University.

Table 4—Five-point loading beam shear test dimensions

Specimen type	Beam size	Span lengths (in.) 2.5d	Span lengths (in.) 5d	Plate lengths (in.) b_1	Plate lengths (in.) b_2
Southern Pine	1½ by 4	10$^5/_8$	21¼	8	12
	2½ by 5½ LQ	13¾	27½	10	12
	2½ by 5½ HQ	13¾	27½	10	12
	3 by 11	27½	55	15	20
	5 by 22	55	110	24	30
Douglas-fir	1½ by 5½ LQ	12½	25	8	12
	1½ by 5/2 HQ	12½	25	10	12
	2½ by 6½ LQ	16¼	32½	10	12
	2/2 by 6½ HQ	16¼	32½	15	20
	3/8 by 12	30	60	15	20
	4$^3/_8$ by 11½ LQ	30	60	15	20
	4$^3/_8$ by 1½ HQ	30	60	15	20
	5$^1/_8$ by 24	60	120	24	30

Table 5—Third-point bending test dimensions

Specimen type	Beam size	Span lengths (in.) 5d	15d
Southern Pine	1½ by 4	21¼	63¾
	2½ by 5½ LQ	27½	82½
	2½ by 5½ HQ	27½	82½
	3 by 11	55	165
	5 by 22	110	330
Douglas-fir	1½ by 5½ LQ	25	75
	1½ by 5½ HQ	25	75
	2½ by 6½ LQ	32½	97½
	2½ by 6½ HQ	32½	97½
	3 1/8 by 12	60	180
	4 3/8 by 11½ LQ	60	180
	4 3/8 by 11½ HQ	60	180
	5 1/8 by 24	120	360

Table 6—Loading rate and load cell accuracies for all tests

Specimen type	Beam size	Beam shear tests: Load rate (in/min)	Beam shear tests: Load accuracy[a] (lb)	Bending tests: Load rate (in/min)	Bending tests: Load accuracy[a] (lb)
Southern Pine	1½ by 4	0.06	50 (0.1)	0.15	20 (0.1)
	2½ by 5½ LQ	0.06	50 (0.1)	0.23	20 (0.1)
	2½ by 5½ HQ	0.06	50 (0.1)	0.23	20 (0.1)
	3 by 11	0.09	200 (0.2)	0.40	100 (0.2)
	5 by 22	0.12	1,000 (0.2)	0.69	200 (0.2)
Douglas-fir	11/2by5'/2LQ	0.04	50 (0.1)	0.10	20 (0.1)
	1½ by 5½ HQ	0.04	50 (0.1)	0.10	20 (0.1)
	2/2 by 6½ LQ	0.06	50 (0.1)	0.15	20 (0.1)
	2½ by 6½ HQ	0.06	50 (0.1)	0.15	20 (0.1)
	3/8 by 12	0.09	200 (0.2)	0.22	100 (0.2)
	4 3/8 by 11½ LQ	0.09	200 (0.2)	0.40	50 (0.1)
	4 3/8 by 11½ HQ	0.09	200 (0.2)	0.40	50 (0.1)
	5 1/8 by 24	0.12	1,000 (0.2)	0.55	200 (0.2)

[a] Percentage of load accuracy divided by maximum load (sensitivitity).

Table 7—Southern Pine beam shear strength results

Beam size	Number of SF[a]/ total number of tests	Moisture content (%)	Shear strength (lb/in²)	COV[b] (%)	Shear at failure (lb/in²)	COV (%)
1½ by 4	12/22	10.0	1,980	10.7	1,970	9.1
2½ by 5½ LQ	22/39	10.1	1,640	9.8	1,660	9.0
2½ by 5½ HQ	28/37	10.2	1,620	12.3	1,630	11.4
3 by 11	20/20	—	1,340	8.6	1,310	10.9
5 by 22	20/20	10.5	970	10.4	970	10.4

[a] = Beam shear failures.
[b] COV is coefficient of variation.

Table 8—Southern Pine flexural strength results

Beam size	Number of tests	Average moisture content (%)	Modulus of rupture (lb/in²)	COV[a] (%)
1½ by 4	23	10.1	10,800	18.8
2½ by 5½ LQ	39	10.1	7,870	23.4
2½ by 5½ HQ	40	10.2	10,500	16.7
3 by 11	20	(b)	8,680	10.3
5 by 22	20	10.6	7,680	15.2

[a] COV is coefficient of variation.
[b] Moisture content not recorded.

Table 9—Southern Pine ASTM shear block results

Beam size	Number of tests	Specific gravity	Moisture content	Shear strength (lb/in²)	Shear strength COV[a]
1½ by 4	12	0.54	10.5	1,380	10.8
2½ by 5½ LQ	17	0.49	10.9	1,360	18.5
2½ by 5½ HQ	27	0.57	11.3	1,390	16.8
3 by 11	20	0.53	10.9	1,420	12.9
5 by 22	20	0.49	10.8	1,340	14.4
(Forest Products Laboratory 1987)[b]		0.51		1,390	14

[a] COV is coefficient of variation
[b] Loblolly pine.

Table 10—Douglas-fir beam shear strength results

Beam size	Number of SF[a]/ total number of tests	Moisture content (%)	Shear strength (lb/in^2)	COV (%)	Shear stress at failure (lb/in^2)	COV[b] (%)
1½ by 5½ LQ	26/31	10.8	1,450	9.7	1,430	9.3
1½ by 5½ HQ	28/32	10.7	1,450	8.3	1,430	8.7
2½ by 6½ LQ	14/19	12.0	1,240	7.8	1,250	7.8
2½ by 6½ HQ	32/37	12.1	1,280	10.5	1,280	10.0
3$^1/_8$ by 12	20/20	11.3	1,000	6.7	1,000	6.7
4$^3/_8$ by 11½ LQ	15/16	9.7	949	9.0	950	8.7
4$^3/_8$ by 11½ HQ	16/17	10.1	936	10.7	940	10.4
5$^1/_8$ by 24	20/20	11.3	770	8.7	770	8.7

[a] = Beam shear failure.
[b] COV is coefficient of variation.

Table 11—Douglas-fir flexural strength results

Beam Size	Number of tests	Average moisture content (%)	Modulus of rupture (lb/in^2)	COV[a]
1½ by 5½ LQ	30	10.7	6,850	24.2
1½ by 5½ HQ	26	10.6	7,470	22.4
2/$_2$ by 6½ LQ	14	11.8	5,360	17.2
2/$_2$ by 6½ HQ	37	11.9	7,900	19.9
3$^1/_8$ by 12	20	11.4	7,170	15.5
4$^3/_8$ by 11½ LQ	16	9.8	5,050	23.2
4$^3/_8$ by 11½ HQ	17	10.0	6,730	23.8
5$^1/_8$ by 24	20	11.2	5,630	13.1

[a] COV is coefficient of variation.

Table 12—Douglas-fir ASTM shear block results

Beam size	Number of tests	Specific gravity	Moisture content	Shear strength (lb/in^2)	Shear strength COV[a]
1½ by 5½ LQ	25	0.44	10.8	1,120	11.6
1½ by 5½ HQ	28	0.49	10.7	1,230	11.2
2½ by 6½ LQ	15	0.42	11.2	1,230	20.3
2½ by 6½ HQ	32	0.46	10.9	1,310	9.5
3$^1/_8$ by 12	20	0.44	11.2	1,110	16.4
4$^3/_8$ by 11½ LQ[b]	15	0.46	10.4	1,590	22.5
4$^3/_8$ by 11½ HQ[b]	16	0.45	10.4	1,540	12.2
5$^1/_8$ by 24	20	0.42	11.4	960	27.7
(Forest Products Laboratory 1987)		0.48[c] 0.46[d]		1,130 1,560	14

[a] COV is coefficient of variation.
[b] Washington State beams.
[c] Coast Douglas-fir.
[d] Interior South Douglas-fir.

Table 13—Regression equations relating shear strength to modulus of rupture

Regressed Equation	Coefficient of Determination (r^2)					
	All Douglas-fir beams	Censored Douglas-fir beam	All Southern Pine beams	Censored Southern Pine beams	Combined data set[a]	Combined censored data set
$y = a+b(X)$	0.10	0.19	0.15	0.23	0.24	0.34
$y = a+b(X)+c(X^2)$	0.09	0.19	0.14	0.26	0.24	0.34
$y = b(X)+c(X^2)$	0.08	0.20	0.13	0.26	0.21	0.33
$y = a(X^b)$	0.09	0.19	0.13	0.24	0.21	0.33
$y = ae^{bx}$	0.09	0.18	0.13	0.22	0.22	0.32
$y = a+b\ln(x)$	0.10	0.20	0.15	0.25	0.23	0.34

[a] Southern Pine and Douglas-fir data sets combined.

Table 14—Analysis of variance for beam shear strength

Specimen type	Beam size	Shear area (in^2)	Mean shear strength (lb/in^2)	Tukey grouping
Southern Pine	ASTM Shear Block[a]	4	2,760	A
	1½ by 4	36.4	1,980	B
	2½ by 5½ LQ	81.8	1,640	C
	2½ by 5½ HQ	81.1	1,620	C
	3 by 11	206.3	1,310	D
	5 by 22	550.0	970	F
Douglas-fir	ASTM Shear Block[a]	4	2,290	A
	1½ by 5½ LQ	35.2	1,450	B
	1 /₂ by 5½ HQ	35.2	1,450	B
	2½ by 6½ LQ	76.3	1,280	B
	2½ by 6½ HQ	75.9	1,240	B C
	3¹/₈ by 12	184.6	1,000	D C
	4³/₈ by 11½	258.4	940	D
	5¹/₈ by 24	607.5	770	D

[a] Shear strength value multiplied by 2 to account for stress concentration.

Tabla 15—Regression results relating shear strength to beam size

Equation	Type of glued-laminated beam	Coefficient of determination (r^2) Beam depth	Shear area	Beam volume
$y = a+bx$	Southern Pine	0.69	0.65	0.59
	Douglas-fir	0.67	0.66	0.49
$y=a+bx+cx^2$	Southern Pine	0.74	0.75	0.72
	Douglas-fir	0.81	0.81	0.80
$y=a+b\ln(x)$	Southern Pine	0.75	0.77	0.77
	Douglas-fir	0.78	0.82	0.82
$y = ae^{bx}$	Southern Pine	0.76	0.73	0.67
	Douglas-fir	0.73	0.73	0.57
$y = ax^b$	Southern Pine	0.79	0.81	0.80
	Douglas-fir	0.82	0.84	0.84

Appendix A—Southern Pine Glued-Laminated Beam Results

Appendix A contains tables of the results of the dimensions, mechanical properties (stiffness and strength), and failure type for test Southern pine beams.

Table A1—Bending and shear data for 5 by 22 Southern Pine glued-laminated beams

			Third-point bending[a]			Five-point beam shear		
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ Ib/in^2)	MOR (lb/in^2)	EJ in TL[b]	EJ failure	Shear stress (lb/in^2)	Type of failure
03-01	5	22	2.22	10,500	Yes	No	1,040	Shear
03-02	5	22	1.95	7,323	Yes	Yes	980	Shear
03-03	5	22	2.28	9,409	No	No	1,050	Shear
03-04	5	22	2.27	8,209	Yes	Yes	1,010	Shear
03-05	5	22	2.24	7,909	Yes	Yes	860	Shear
03-06	5	22	1.99	9,477	Yes	Yes	1,080	Shear
03-07	5	22	1.94	6,573	Yes	Yes	900	Shear
03-08	5	22	2.07	7,991	Yes	Yes	930	Shear
03-09	5	22	1.63	6,559	Yes	Yes	9.70	Shear
03-10	5	22	1.84	6,709	Yes	Yes	1,040	Shear
03-11	5	22	2.22	6,695	Yes	No	950	Shear
03-12	5	22	1.95	7,009	Yes	Yes	1,140	Shear
03-13	5	22	2.28	8,127	Yes	Yes	1,100	Shear
03-14	5	22	2.27	6,162	Yes	Yes	740	Shear
03-15	5	22	2.24	6,600	Yes	Yes	980	Shear
03-16	5	22	1.99	8,114	Yes	Yes	970	Shear
03-17	5	22	1.94	8,332	Yes	Yes	950	Shear
03-18	5	22	2.07	5,932	Yes	Yes	950	Shear
03-19	5	22	1.93	8,114	Yes	No	970	Shear
03-20	5	22	1.84	7,786	Yes	Yes	760	Shear

[a] MOE = modulus of elasticity, MOR = modulus of rupture, EJ = end (finger) joint, TL = tension lamination.

[b] EJ in TL is defined by the critical region with 85 percent or more of the maximum moment (critical region is 12 ft).

Table A2—Bending and shear data for 3 by 11 Southern Pine glued-laminated beams

			Third-point bending[a]			Five-point beam shear		
Beam	Depth (in.)	Width (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	EJ in TL[b]	EJ failure	Shear stress (lb/in^2)	Type of failure
02-01	3.00	11.00	1.88	9,673	No	No	1,340	Shear
02-02	3.00	11.00	2.03	8,864	Yes	No	1,290	Shear
02-03	3.00	11.00	1.95	6,964	Yes	No	1,220	Shear
02-04	3.00	11.00	1.77	7,836	Yes	No	1,450	Shear
02-05	3.00	11.00	1.81	8,864	Yes	Yes	1,280	Shear
02-06	3.00	11.00	1.97	7,382	Yes	Yes	1,340	Shear
02-07	3.00	11.00	1.95	8,236	Yes	No	1,470	Shear
02-08	3.00	11.00	1.94	8,800	Yes	Yes	1,290	Shear
02-09	3.00	11.00	1.79	10,227	No	No	1,430	Shear
02-10	3.00	11.00	1.90	9,445	Yes	Yes	1,220	Shear
02-11	2.98	10.98	1.84	6,970	Yes	No	1,261	Shear
02-12	2.99	11.00	2.20	9,705	Yes	Yes	925	Shear
02-13	2.99	11.00	1.73	9,850	No	No	1,540	Shear
02-14	2.99	11.00	2.11	7,164	Yes	Yes	1,360	Shear
02-15	3.00	10.99	2.16	10,227	No	No	1,170	Shear
02-16	3.00	10.99	1.96	8,877	Yes	No	1,100	Shear
02-17	3.00	10.98	2.07	10,091	Yes	Yes	1,420	Shear
02-18	3.00	10.96	1.92	8,364	Yes	Yes	1,340	Shear
02-19	3.00	10.98	1.92	6,036	Yes	Yes	1,470	Shear
02-20	3.00	10.99	2.00	8,359	Yes	No	1,340	Shear

[a] MOE = modulus of elasticity, MOR =modulus of rupture, EJ = end (finger) joint, TL = tension lamination.

[b] EJ in TL is defined by the critical region with 85 percent or more of the maximum moment (critical region is 6 ft).

Table A—Bending and shear data for 2½ by 5½, HQ Southern Pine glued-laminated beams

		Third-point bending[a]				Five-point beam shear			
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	Typeof failure	Depth (in.)	Width (in.)	Shear stress (lb/in^2)	Type of failure[b]
01-011	2.35	5.45	2.31	12,187	Bending	2.36	5.36	1,724	Shear
01-012	2.36	5.44	2.35	12,035	Bending	2.37	5.38	1,210	Shear
01-021	2.36	5.43	2.23	11,122	Bending	2.38	5.35	1,681	Shear
01-031	2.37	5.44	2.03	9,259	Bending	2.38	5.37	1,549	Rotation
01-032	2.37	5.44	2.08	12,991	Bending	2.38	5.44	1,585	Shear
01-041	2.37	5.44	2.29	13,383	Bending	2.38	5.45	1,570	Shear
01-042	2.38	5.44	2.20	13,313	Bending	2.36	5.37	1,641	Shear
01-051	2.38	5.45	2.19	10,684	Bending	2.30	5.38	1,254	Shear
01-052	2.35	5.44	1.83	11,863	Bending	2.37	5.36	1,729	Shear
01-061	2.38	5.44	2.44	10,677	Bending	2.36	5.38	1,618	Shear
01-062	2.37	5.44	1.81	7,576	Bending	2.36	5.38	1,494	Rotation
01-071	2.37	5.44	1.75	7,039	Bending	2.38	5.38	1,417	Shear
01-081	2.37	5.44	2.03	11,674	Bending	2.37	5.38	1,471	Shear
01-082	2.37	5.44	1.62	10,268	Bending	2.26	5.36	2,045	Shear
01-091	2.36	5.44	1.61	9,090	Bending	2.36	5.38	1,455	Shear
01-092	2.36	5.44	1.65	10,249	Bending	2.36	5.36	1,618	Shear
01-101	2.37	5.44	1.81	11,622	Bending	2.37	5.37	1,673	Shear
01-102	2.37	5.44	2.00	9,498	Bending	2.37	5.37	1,888	Shear
01-111	2.37	5.44	2.12	10,123	Bending	2.25	5.38	1,620	Bending
01-112	2.38	5.43	1.80	8,437	Bending	2.38	5.39	1,772	Shear
01-121	2.37	5.44	1.67	11,183	Bending	2.37	5.39	1,834	Bending
01-122	2.38	5.44	1.42	12,902	Bending	2.38	5.38	1,729	Shear
01-131	2.36	5.44	1.42	9,799	Bending	2.39	5.50	1,828	Shear
01-132	2.38	5.44	1.46	9,424	Bending	2.38	5.44	1,633	Bending
01-141	2.38	5.44	1.97	10,422	Bending	2.36	5.38	1,946	Bending
01-142	2.37	5.44	2.30	11,395	Bending	2.39	5.41	1,444	Shear
01-151	2.37	5.44	1.50	10,929	Bending	2.38	5.46	1,778	Shear
01-152	2.37	5.44	1.87	7,351	Bending	2.38	5.37	1,460	Shear
01-161	2.37	5.45	1.94	13,961	Bending	2.38	5.39	1,867	Shear
01-171	2.37	5.45	1.79	8,352	Bending	2.38	5.38	1,732	Shear
01-172	2.37	5.44	1.63	8,707	Bending	2.36	5.37	1,829	Shear
01-181	2.30	5.45	1.81	11,414	Bending	2.38	5.38	1,646	Shear
01-182	2.37	5.45	1.67	8,824	Bending	2.38	5.40	1,385	Shear
01-191	2.37	5.44	1.78	8,489	Bending	2.40	5.37	1,608	Rotation
01-192	2.38	5.45	2.04	9,411	Bending	2.38	5.39	1,601	Bending
01-201	2.37	5.44	1.75	9,717	Bending	2.38	5.39	1,368	Shear
01-202	2.37	5.44	1.57	12,007	Bending	2.36	5.37	1,712	Rotation

[a] MOE = modulus of elasticity, MOR = modulus of rupture.
[b] Rotation of specimen in test frame caused by excessive crushing.

Table A4—Bending and shear data for 2½ by 5½, LQ Southern Pine glued-laminated beams

	Third-point bending[a]					Five-point beam shear			
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	Type of failure[b]	Width (in.)	Depth (in.)	Shear stress (lb/in^2)	Type of failure[c]
01-01T	2.38	5.45	1.73	9,490	Bending	2.39	5.75	1,748	Shear
01-01B	2.38	5.45	1.68	8,438	Bending	2.40	5.75	1,505	Shear
01-02T	2.39	5.45	1.98	10,069	Bending	2.39	5.76	1,601	Shear
01-02B	2.38	5.45	1.65	7,728	Bending	2.38	5.76	1,656	Bending
01-03T	2.37	5.45	1.75	9,113	Bending	2.40	5.76	1,706	Bending
01-03B	2.38	5.44	2.09	10,184	CP	2.39	5.74	1,602	Shear
01-04T	2.38	5.44	2.05	6,527	Bending	2.38	5.69	1,580	Shear
01-04B	2.37	5.45	1.85	9,622	Bending	2.39	5.74	1,574	Shear
01-05T	2.39	5.45	1.31	8,031	Bending	2.39	5.68	1,886	Rotation
01-05B	2.38	5.44	1.59	6,190	Bending	2.38	5.68	1,755	Bending
01-06T	2.38	5.44	1.43	9,718	Bending	2.38	5.37	1,896	Shear
01-060	2.37	5.44	1.38	9,405	Bending	2.38	5.35	1,887	Shear
01-07T	2.37	5.45	1.83	11,042	CP	2.39	5.75	1,647	Shear
01-07B	2.37	5.45	1.58	6,630	Bending	2.41	5.75	1,646	Shear
01-08B	2.38	5.44	1.00	4,977	Bending	2.39	5.75	1,880	Rotation
01-09T	2.39	5.44	1.36	6,421	Bending	2.38	5.75	1,702	Bending
01-09B	2.39	5.44	1.75	7,144	Bending	2.37	5.75	1,346	Shear
01-10T	2.38	5.44	1.17	7,373	Bending	2.39	5.80	1,396	Shear
01-10B	2.39	5.45	1.49	7,940	Bending	2.39	5.75	1,562	Bending
01-11T	2.37	5.44	1.69	5,464	Bending	2.39	5.77	1,583	Bending
01-11B	2.37	5.44	1.77	5,959	Bending	2.40	5.73	1,717	Bending
01-12T	2.37	5.45	1.58	8,245	Bending	2.39	5.74	1,727	Shear
01-12B	2.32	5.45	1.31	12,146	CP	2.38	5.36	1,964	Shear
01-13T	2.32	5.44	1.65	9,185	Bending	2.37	5.38	1,615	Shear
01-13B	2.37	5.44	1.83	9,666	Bending	2.37	5.35	1,610	Bending
01-14T	2.38	5.45	2.09	5,075	Bending	2.37	5.37	1,391	Shear
01-14B	2.38	5.44	2.08	5,911	Bending	2.38	5.39	1,724	Shear
01-15T	2.38	5.45	0.97	7,564	Bending	2.37	5.38	1,731	Rotation
01-15B	2.37	5.45	1.36	9,185	Bending	2.38	5.36	1,697	Bending
01-16T	2.38	5.45	1.55	5,287	Bending	2.39	5.76	1,510	Shear
01-16B	2.38	5.45	1.64	7,124	Bending	2.39	5.76	1,337	Bending
01-17T	2.37	5.45	1.57	9,484	Bending	2.40	5.75	1,790	Bending
01-17B	2.36	5.45	1.80	9,279	Bending	2.39	5.76	1,709	Bending
01-18T	2.38	5.45	1.64	8,645	Bending	2.39	5.37	1,542	Bending
01-18B	2.33	5.44	1.49	6,379	Bending	2.34	5.75	1,705	Shear
01-19T	2.37	5.45	1.60	4,427	Bending	2.39	5.76	1,556	Bending
01-19B	2.37	5.44	1.63	7,375	Bending	2.38	5.74	1,725	Shear
01-20T	2.37	5.45	1.77	9,131	Bending	2.39	5.75	1,658	Shear
01-20B	2.37	5.45	1.69	5,331	Bending	2.39	5.75	1,535	Shear

[a] MOE = modulus of elasticity, MOR = modulus of rupture.
[b] CP =compression parallel to grain failure.
[c] R = Rotation of specimen in test frame caused by excessive crushing.

Table A5—Bending and shear data for 1½ by 4 Southern Pine glued-laminated beams

	Third-point bending[a]					Five-point beam shear			
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	Type of failure	Width (in.)	Depth (in.)	Shear stress (lb/in^2)	Typeof failure[b]
00-03L	1.31	4.13	1.90	11,144	Bending	1.38	4.12	2,076	Shear
00-04L	1.39	4.14	1.97	12,308	Bending	1.32	4.12	1,996	Rotation
00-05L	1.38	4.13	2.03	12,587	Bending	1.39	4.12	1,981	Rotation
00-05R	1.40	4.15	1.70	11,246	Bending	1.31	4.13	1,851	Bending
00-06R	1.38	4.14	2.14	12,380	Bending	1.39	4.13	1,751	Shear
00-07R	1.38	4.14	2.48	7,687	Bending	1.38	4.13	2,054	Shear
00-07R2	1.39	4.13	1.84	9,954	Bending	1.38	4.12	1,811	Shear
00-08L	1.31	4.14	1.65	6,576	Bending	1.38	4.14	1,819	Shear
00-09R	1.39	4.13	1.78	11,016	Bending	1.40	4.12	1,698	Shear
00-11L	1.40	4.14	1.94	11,473	Bending	1.38	4.13	2,027	Rotation
00-11R	1.38	4.14	1.68	10,112	Bending	1.32	4.12	2,448	Shear
00-12R	1.38	4.13	1.96	10,904	Bending	1.38	4.12	2,060	Bending
00-14R	1.38	4.13	1.92	7,889	Bending	1.38	4.14	2,023	Shear
00-14R2	1.38	4.14	2.14	12,998	Bending	1.38	4.12	2,080	Rotation
00-15R	1.38	4.15	1.81	12,517	Bending	1.38	4.12	1,877	Shear
00-15R2	1.38	4.13	1.79	12,390	Bending	1.38	4.14	2,142	Shear
00-16L	1.31	4.13	1.59	5,645	Bending	1.38	4.14	1,927	Shear
00-17R	1.38	4.13	1.97	11,940	Bending	1.38	4.12	1,846	Bending
00-17R2	1.38	4.13	2.02	12,301	Bending	1.41	4.14	1,826	Rotation
00-19R	1.39	4.13	1.85	11,898	Bending	1.38	4.14	2,162	Shear
00-20R	1.39	4.13	2.16	9,986	Bending	1.38	4.13	1,716	Rotation
00-20R2	1.38	4.14	2.33	12,158	Bending	1.38	4.12	2,199	Rotation

[a] MOE = modulus of elasticity, MOR = modulus of rupture.
[b] R = Rotation of specimen in test frame caused by excessive crushing.

Appendix B—Douglas-fir Glued-Laminated Beam Results

Appendix B contains tables of the results of the dimensions, mechanical properties (stiffness and strength), and failure type for test Douglas-fir beams.

Table B1—Bending and shear data for 5 by 24 Douglas-fir glued-laminated beams

			Third-point bending[a]			Five-point beam shear		
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	EJ in TL[b]	EJ failure	Shear stress (lb/in^2)	Type of failure
501	5.00	23.94	1.86	5,818	Yes	Yes	652	Shear
502	5.06	23.88	1.83	5,464	Yes	Yes	661	Shear
503	5.06	23.81	1.78	5,016	Yes	Yes	836	Shear
504	5.06	23.81	—	6,421	Yes	Yes	817	Shear
505	5.06	23.94	1.67	5,324	Yes	Yes	724	Shear
506	5.06	23.94	1.75	5,994	Yes	Yes	794	Shear
507	5.06	23.81	1.88	6,270	Yes	No	813	Shear
508	5.06	23.75	1.94	6,909	No	No	695	Shear
509	5.06	23.88	1.88	5,689	Yes	Yes	756	Shear
510	5.06	23.81	1.86	5,769	Yes	Yes	734	Shear
511	5.06	23.88	1.85	5,127	Yes	Yes	682	Shear
512	5.06	23.88	1.68	6,687	Yes	Yes	774	Shear
513	5.06	23.88	1.74	6,100	Yes	Yes	832	Shear
514	5.06	23.84	1.82	5,979	Yes	Yes	704	Shear
515	5.06	23.84	1.87	3,903	Yes	Yes	829	Shear
516	5.06	23.88	1.85	5,215	No	No	697	Shear
517	5.06	23.81	1.88	5,719	Yes	No	893	Shear
518	5.06	23.78	1.77	5,042	Yes	No	815	Shear
519	5.06	23.88	1.82	5,938	Yes	Yes	804	Shear
520	5.06	23.88	1.78	4,167	Yes	No	818	Shear

[a] MOE = modulus of elasticity, MOR = modulus of rupture, EJ = end (finger) joint, TL = tension lamination.

[b] EJ in TL is defined by the critical region with 85 percent or more of the maximum moment (critical region is 12 ft).

Table B2—Bending and shear data for 4³/₈ by 11³/₈ HQ Douglas-fir glued-laminated beams

	Third-point bending[a]					Five-point beam shear			
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	Type of failure	Width (in.)	Depth (in.)	Shear stress (lb/in^2)	Type of failure
04-01EB	4.43	11.60	1.66	7,093	Bending	4.30	11.81	815	Shear
04-01WB	4.43	11.95	1.64	5,270	Bending	4.32	12.05	904	Shear
04-02EB	4.39	11.55	1.65	8,755	Bending	4.24	11.75	791	Shear
04-02WB	4.39	11.60	1.66	6,245	Bending	4.32	11.62	855	Shear
04-03EB	4.36	11.43	1.85	7,778	Bending	4.32	11.58	1,134	Shear
04-03WB	4.37	12.00	1.69	7,466	Bending	4.31	12.01	1,047	Shear
04-04EB	4.36	11.95	1.95	8,093	Bending	4.32	12.14	864	Shear
04-04WB	4.40	12.06	1.58	4,834	Bending	4.29	12.14	937	Shear
04-06EB	4.41	11.91	2.18	8,482	Bending	4.28	11.69	934	Shear
04-06WB	4.36	11.55	1.86	7,527	Bending	4.36	11.79	835	Shear
04-07WB	4.35	11.91	2.09	5,945	SP	4.33	11.93	1,004	Shear
04-08WB	4.37	12.04	2.00	2,655	SP	4.16	12.20	979	Shear
04-09EB	4.50	11.45	2.07	8,635	Bending	4.21	11.28	1,088	Shear
04-09WB	4.39	11.54	2.26	5,139	Bending	4.21	11.53	939	Shear
04-14EB	4.37	11.85	1.62	7,357	Bending	4.37	11.81	999	Bending
04-15EB	4.44	11.87	1.90	6,696	Bending	4.25	11.96	992	Shear
04-15WB	4.33	11.66	1.59	6,480	Bending	4.36	11.86	856	Shear

[a] MOE = modulus of elasticity, MOR = modulus of rupture, SP = shear failure by propagation of a existing split,

Table B3—Bending and shear data for 4 3/8 by 11 3/8 LQ Douglas-fir glued-laminated beams

	Third-point bending[a]					Five-point beam shear			
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	Type of failure	Width (in.)	Depth (in.)	Shear stress (lb/in^2)	Type of failure
04-01ET	4.38	11.63	1.39	3,784	Bending	4.33	11.42	996	Shear
04-01WT	4.28	11.68	1.49	4,261	Bending	4.41	11.68	1,076	Shear
04-02ET	4.15	11.60	—	—		4.51	11.73	934	Shear
04-02WT	4.33	11.87	1.72	6,452	Bending	4.21	11.83	1,044	Shear
04-03ET	4.31	11.90	1.49	5,229	Bending	4.27	11.93	966	Bending
04-03WT	4.32	11.36	1.67	5,441	Bending	4.28	11.39	1,055	Shear
04-04ET	4.37	11.50	1.60	6,417	Bending	4.35	11.32	1,011	Shear
04-04WT	4.33	11.38	1.55	5,090	Bending	4.30	11.38	809	Shear
04-06WT	4.38	11.70	1.63	4,566	Bending	4.33	11.42	936	Shear
04-07WT	4.37	11.50	1.86	7,760	Bending	4.25	11.36	954	Shear
04-08WT	4.38	11.48	1.88	4,719	Bending	4.31	11.34	965	Shear
04-09ET	4.40	12.11	1.65	4,575	Bending	4.26	12.22	784	Shear
04-09WT	4.40	12.02	1.97	4,963	Bending	4.18	11.87	951	Shear
04-14ET	4.40	11.50	1.34	3,528	Bending	4.33	11.44	873	Shear
04-15ET	4.42	11.41	1.67	3,437	Bending	4.25	11.38	877	Shear
04-15WT	4.39	11.52	1.71	5,473	Bending	4.33	11.68	974	Bending

[a] MOE = modulus of elasticity, MOR = modulus of rupture.

Table B4—Bending and shear data for 3 by 12 Douglas-fir glued-laminated beams

			Third-point bending[a]			Five-point beam shear		
Beam	Depth (in.)	Width (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	EJ in TLb	EJ failure	Shear stress (lb/in2)	Type of failure
401	3.06	11.88	2.14	7,502	No	No	930	Shear
402	3.06	11.88	2.47	8,003	Yes	Yes	996	Shear
403	3.06	11.91	2.13	7,712	Yes	Yes	873	Shear
404	3.06	11.91	2.05	7,529	Yes	Yes	950	Shear
405	3.06	11.91	2.31	6,435	Yes	Yes	1,063	Shear
406	3.06	11.91	2.11	7,505	Noo	No	1,024	Shear
407	3.06	11.94	2.31	10,373	N	No	995	Shear
408	3.06	11.94	2.22	8,245	No	No	939	Shear
409	3.06	11.94	2.35	6,764	No	No	1,044	Shear
410	3.09	11.94	1.98	6,614	Yes	Yes	1,027	Shear
411	3.06	11.94	2.10	6,946	No	No	1,140	Shear
412	3.06	11.94	2.10	8,150	No	No	961	Shear
413	3.06	11.94	2.18	5,444	Yes	No	990	Shear
414	3.06	11.94	2.05	6,207	Yes	No	942	Shear
415	3.09	11.94	1.98	7,913	Yes	Yes	896	Shear
416	3.09	11.95	1.96	5,563	Yes	Yes	1,140	Shear
417	3.06	11.91	2.12	7,032	Yes	No	984	Shear
418	3.06	11.91	2.47	7,214	No	No	1,004	Shear
419	3.06	11.97	2.26	5,572	No	No	997	Shear
420	3.06	11.94	2.20	6,686	Yes	Yes	1,033	Shear

[a] MOE = modulus of elasticity, MOR = modulus of rupture, EJ = end (finger) joint, TL = tension lamination.

[b] EJ in TL is defined by the critical region with 85 percent or more of the maximum moment (critical region is 6 ft).

Table B5—Bending and shear data for 2½ by 6½, HQ Douglas-fir glued-laminated beams

Beam	Third-point bending[a]					Five-point beam shear			
	Width (in.)	Depth (in.)	MOE ($\times 10^6$ lb/in^2)	MOR (lb/in^2)	Typeof failure	Width (in.)	Depth (in.)	Shear stress (lb/in^2)	Type of failure[b]
3011	2.50	6.25	1.88	7,800	Bending	2.45	6.13	1,150	Shear
3012	2.50	6.25	2.06	6,177	Bending	2.45	6.16	994	Shear
3021	2.50	6.25	1.90	5,366	Bending	2.42	6.08	1,226	Shear
3022	2.44	6.38	1.45	9,062	Bending	2.44	6.19	1,227	Shear
3031	2.38	6.13	-	7,544	Bending	2.44	6.17	1,457	Shear
3032	2.50	6.25	2.05	9,216	Bending	2.44	6.19	1,289	Shear
3041	2.50	6.25	2.10	8,071	Bending	2.47	6.19	1,428	Shear
3051	2.50	6.19	2.00	9,359	Bending	2.44	6.19	1,238	Shear
3052	2.44	6.13	1.77	7,924	Bending	2.44	6.16	1,429	Shear
3061	2.47	6.19	1.95	8,096	Bending	2.44	6.19	1,306	Bending
3062	2.50	6.25	1.81	5,320	Bending	2.45	6.17	1,250	Shear
3071	2.44	6.19	2.03	9,372	Bending	2.44	6.52	1,224	Shear
3072	2.44	6.38	1.57	7,493	Bending	2.44	6.19	1,077	Shear
3081	2.47	6.19	2.26	9,445	CP	2.47	6.17	1,206	Shear
3082	2.50	6.25	2.09	5,348	Bending	2.44	6.19	961	Shear
3091	2.47	6.13	2.36	9,488	CP	2.44	6.06	1,315	Rotation
3092	2.47	6.13	2.28	10,511	CP	2.44	6.13	1,347	Shear
3101	2.50	6.25	2.05	9,649	Bending	2.44	6.19	1,576	Shear
3102	2.50	6.25	2.07	7,305	Bending	2.44	6.20	1,483	Shear
3111	2.44	6.13	2.14	10,229	CP	2.42	6.09	1,291	Shear
3112	2.44	6.16	1.95	7,229	Bending	2.42	6.08	1,252	Shear
3121	2.56	6.19	2.06	6,150	Bending	2.44	6.06	1,466	Rotation
3122	2.50	6.13	1.92	5,531	Bending	2.44	6.06	1,169	Shear
3141	2.50	6.19	2.20	7,153	Bending	2.47	6.19	1,362	Shear
3142	2.44	6.38	1.88	8,025	Bending	2.45	6.17	1,355	Shear
3151	2.50	6.19	2.26	9,322	Bending	2.44	6.09	1,293	Shear
3152	2.47	6.13	2.13	9,157	Bending	2.44	6.19	1,258	Shear
3161	2.47	6.19	2.04	7,817	Bending	2.47	6.11	1,415	Shear
3162	2.44	6.06	2.22	8,030	Bending	2.44	6.08	1,333	Shear
3171	2.44	6.13	2.42	9,749	Bending	2.47	6.09	1,117	Shear
3172	2.44	6.13	1.79	8,741	Bending	2.44	6.11	1,368	Shear
3181	2.50	6.13	2.04	5,859	Bending	2.44	6.09	1,198	Rotation
3182	2.47	6.13	1.93	6,490	Bending	2.45	6.09	1,207	Shear
3191	2.44	6.19	1.69	9,467	Bending	2.44	6.08	1,315	Rotation
3192	2.47	6.19	2.13	5,258	Bending	2.44	6.13	1,340	Shear
3201	2.44	6.13	2.01	5,763	Bending	2.44	6.09	1,173	Shear
3202	2.47	6.09	1.97	9,710	CP	2.42	6.09	1,411	Shear

[a] MOE =modulus of elasticity, MOR = modulus of rupture, CP = compression parallel to grain failure.
[b] R = rotation of specimen in test frame caused by excessive crushing.

Table B6—Bending and shear data for 2½ by 6½, LQ Douglas-fir glued-laminated beams

	Third-point bending[a]					Five-point beam shear			
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	Type of failure	Width (in.)	Depth (in.)	Shear Stress (lb/in^2)	Type of failure[b]
2011	—	—	—	—		2.44	6.50	1,220	Bending
2031	2.50	6.50	1.45	6,078	Bending	2.44	6.50	1,300	Shear
2041	2.47	6.47	1.53	6,235	Bending	2.42	6.50	1,389	Shear
2051	2.47	6.50	1.15	5,570	Bending	2.44	6.50	1,334	Shear
2061	—	—	—	—	—	2.44	6.47	1,485	Bending
2071	2.47	6.50	1.66	3,814	Bending	2.44	6.50	1,155	Shear
2081	2.50	6.50	1.66	5,586	Bending	2.42	6.47	1,175	Shear
2091	2.50	6.50	1.76	5,753	Bending	2.44	6.48	1,360	Shear
2101	2.56	6.56	1.44	4,853	Bending	2.44	6.50	1,207	Shear
2111	2.50	6.50	1.40	6,161	Bending	2.44	6.50	1,240	Shear
2121	—	—	—	—	—	2.42	6.47	1,287	Rotation
2131	—	—	—	—	—	2.44	6.50	1,225	Bending
2141	2.50	6.50	1.62	4,328	Bending	2.44	6.50	1,292	Shear
2151	2.50	6.50	1.66	7,412	Bending	2.41	6.47	1,186	Shear
2161	2.50	6.50	1.85	4,886	Bending	2.44	6.48	1,152	Shear
2171	2.50	6.50	1.64	5,820	Bending	2.41	6.50	1,315	Shear
2181	2.50	6.50	1.42	3,969	Bending	2.44	6.50	1,175	Shear
2191	2.50	6.50	1.43	5,061	Bending	2.44	6.50	1,053	Shear
2501	—	—	—	—	—	2.44	6.50	1,281	Bending

[a] MOE = modulus of elasticity, MOR = modulus of rupture.
[b] R = rotation of specimen in test frame caused by excessive crushing.

Table B7—Bending and shear data for 1½ by 5, HQ Douglas-fir glued-laminated beams

	Third-point bending test[a]					Five-point beam shear test			
Beam	Width (in.)	Depth (in.)	MOE ($x10^6$ lb/in^2)	MOR (lb/in^2)	Type of failure	Width (in.)	Depth (in.)	Shear stress (lb/in^2)	Type of failure[b]
1011	1.38	5.13	1.24	3,782	SP	1.38	5.09	1,413	Shear
1012	1.38	5.13	1.64	7,947	Bending	1.38	5.00	1,538	Shear
1021	1.38	5.06	1.59	6,450	Bending	1.38	5.09	1,528	Shear
1022	1.38	5.00	1.77	5,725	Bending	1.38	5.00	1,429	Shear
1031	1.38	5.13	1.89	5,615	SP	1.38	5.13	1,317	Shear
1032	1.38	4.94	1.82	5,159	Bending	1.38	5.00	1,391	Shear
1052	1.38	4.97	1.56	3,932	Bending	1.38	4.97	1,377	Rotation
1061	1.38	5.06	2.19	6,113	Bending	1.38	5.13	1,595	Shear
1062	1.38	4.94	1.54	8,905	Bending	1.38	5.00	1,508	Shear
1071	1.38	5.08	2.57	8,941	Bending	1.38	5.25	1,376	Shear
1081	1.41	5.13	1.63	7,760	Bending	1.38	5.06	1,433	Shear
1082	1.38	4.97	1.66	10,858	Bending	1.38	4.97	1,449	Shear
1101	1.38	5.06	2.17	7,758	Bending	1.38	5.13	1,518	Shear
1102	1.38	5.00	1.87	7,237	Bending	1.38	5.00	1,511	Shear
1111	1.38	5.06	1.54	6,150	Bending	1.38	5.13	1,482	Shear
1121	1.38	5.06	1.66	7,103	Bending	1.38	5.17	1,208	Bending
1122	1.38	4.95	2.35	9,883	Bending	1.38	5.00	1,433	Shear
1132	1.38	4.94	2.24	8,488	Bending	1.38	5.03	1,718	Shear
1141	1.38	5.06	1.95	6,524	Bending	1.38	5.13	1,233	Shear
1142	1.38	4.94	2.05	5,159	Bending	1.38	5.03	1,148	Shear
1151	1.38	5.06	1.68	3,402	SP	1.38	5.09	1,323	Shear
1171	1.38	5.13	2.27	10,204	Bending	1.38	5.13	1,519	Shear
1172	1.38	4.97	1.41	6,889	Bending	1.38	5.03	1,360	Shear
1182	1.38	4.94	2.47	8,787	Bending	1.38	5.03	1,390	Shear
1191	1.38	5.06	2.56	9,160	CP	1.38	5.16	1,345	Shear
1192	1.38	5.00	2.24	6,513	Bending	1.38	5.03	1,588	Bending
1201	1.38	5.06	1.93	4,934	SP	1.38	5.13	1,398	Shear
1019	1.38	5.08	1.92	6,547	Bending	1.38	5.13	1,310	Bending
1202	1.38	4.94	1.77	6,021	Bending	1.38	5.00	1,538	Shear
1013	1.38	5.00	1.89	3,637	Bending	1.38	5.03	1,308	Shear
1113	1.38	5.06	2.02	6,954	Bending	1.38	5.13	1,562	Shear

[a] MOE = modulus of elasticity, MOR = modulus of rupture, CP = compression parallel to grain failure, SP = shear failure by propagation of a existing split

[b] R = Rotation of specimen in test frame caused by excessive crushing

Table B8—Bending and shear data for 1½ by 5, LQ Douglas-fir glued-laminated beams

	Third-point bending[a]					Five-point beam shear			
Beam	Width (in.)	Depth (in.)	MOE (x106 lb/in^2)	MOR (lb/in^2)	Type of failure	Width (in.)	Depth (in.)	Shear stress (lb/in^2)	Type of failure[b]
11	1.38	5.22	1.66	7,484	Bending	1.38	5.25	1,538	Shear
21	1.38	5.22	2.00	8,204	Bending	1.38	5.25	1,454	Rotation
31	1.38	5.25	1.99	3,335	Bending	1.38	5.22	1,624	Shear
41	1.41	5.25	1.94	6,901	Bending	1.38	5.19	1,373	Shear
51	1.38	5.22	1.77	4,830	SP	1.38	5.22	1,384	Shear
52	1.38	5.13	2.07	6,093	Bending	1.38	5.25	1,611	Shear
61	1.38	5.22	2.22	7,782	Bending	1.38	5.16	1,469	Shear
62	1.38	5.13	1.95	8,492	Bending	1.38	5.16	1,425	Shear
71	1.38	5.22	1.99	5,550	Bending	1.38	5.22	1,380	Shear
72	1.38	5.13	2.45	7,875	Bending	1.38	5.19	1,370	Shear
81	1.41	5.25	1.83	5,999	Bending	1.38	5.22	1,431	Shear
82	1.38	5.13	2.11	6,674	Bending	1.38	5.19	1,348	Shear
91	1.41	5.25	1.89	7,647	Bending	1.38	5.25	1,525	Shear
92	1.38	5.13	1.77	10,694	Bending	1.38	5.06	1,504	Shear
101	1.38	5.19	2.58	3,910	Bending	1.38	5.25	1,082	Shear
102	1.38	5.13	1.61	5,002	Bending	1.38	5.16	1,633	Shear
111	1.38	5.22	1.84	6,008	Bending	1.38	5.25	1,550	Shear
121	1.38	5.22	1.58	6,428	SP	1.38	5.22	1,220	Rotation
132	1.38	5.12	1.98	6,135	Bending	1.38	5.16	1,164	Shear
141	1.38	5.22	1.67	7,642	Bending	1.38	5.25	1,311	Shear
151	1.38	5.22	1.43	4,725	Bending	1.38	5.25	1,304	Shear
161	1.38	5.22	2.06	8,799	Bending	1.38	5.22	1,549	Shear
162	1.38	5.13	1.69	7,202	Bending	1.38	5.16	1,595	Shear
171	1.38	5.22	1.75	6,342	Bending	1.38	5.25	1,358	Shear
181	1.41	5.25	1.83	6,169	Bending	1.38	5.25	1,354	Shear
182	1.38	5.13	1.98	5,983	Bending	1.38	5.11	1,483	Bending
191	1.38	5.19	2.07	9,613	Bending	1.38	5.22	1,624	Shear
192	1.38	5.13	1.71	4,637	Bending	1.38	5.16	1,276	Rotation
113	1.38	5.19	2.18	6,699	Bending	1.38	5.25	1,407	Shear
73	1.38	5.16	1.88	8,754	Bending	1.38	5.19	1,406	Shear
201	1.38	5.22	1.93	7,500	Bending	1.38	5.25	1,614	Shear
202	1.38	5.13	1.73	7,202	Bending	1.38	5.19	1,399	Rotation

[a] MOE = modulus of elasticity. MOR = modulus of rupture, SP = shear failure by propagation of a existing split
[b] R = rotation of specimen in test frame caused by excessive crushing.

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