Shear Strength of Glued-Laminated Timber Beams and Panels

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Abstract
Five sizes of glued-laminated timber beams and panels are being tested in a five-point beam-shear test configuration to determine the shear strength capacity and size relationships. A total of 200 matched specimens will be tested: 100 loaded edgewise about the strong axis and 100 loaded flatwise about the weak axis. Statistical methods will be used to estimate mean and coefficient of variation considering censored data, test the significance of strength-size effect, and correlate between beam shear and ASTM D143 block shear. At the time this paper was written, only the beams loaded edgewise about the strong axis were completely tested along with the smaller-sized beams loaded about the weak axis. This paper presents results for the strong axis tests and a general discussion about the weak axis tests. To date, edgewise specimens had similar size–strength relationships as were observed in previous glued-laminated shear studies.

Keywords: Glued-laminated, shear strength, beams, panels, design, size effect

Introduction
Allowable shear strength values for wood construction are determined by adjusting ASTM D143 (1995a) shear-block results, according to methods specified by ASTM D245 (1995b) for visually graded lumber and ASTM D3737 (1995e) for glued-laminated timber material. Researchers have questioned the validity of predicting beam shear by ASTM shear-block testing (Ethington and others 1979; Radcliffe and Suddarth 1955). In 1994, Soltis and Rammer (1994) established a relationship between ASTM D143 shear block and beam shear by considering the size of the specimens. Although their research correlates beam shear to traditional testing methods, questions remain about shear design. One such question concerns the design of vertically laminated deck panels for timber bridges.

Deck panels loaded flatwise about the weak axis have a shear design value less than panels loaded edgewise about the strong axis (AITC 1993; AFPA 1991). When you consider the current ASTM standards, past and current shear research, and laminating effects, the difference in shear strength seems counter intuitive.

Objective
The general objective of this study is to improve the shear design criteria as it applies to deck panels. Specific objectives include the following:

- Improve the beam-shear database to include axial glued-laminated deck combinations.
- Determine if there is a correlation between shear strength and size for glued-laminated deck panels. Size parameters considered are beam width, shear area, or volume.

- Determine if the shear strength versus shear area equation derived for horizontally laminated beams is applicable to vertically laminated beams.
- Clarify the design method for highway bridge decks by investigation of the failure mode in panels bent about the weak axis.

These objectives will be met through an experimental testing program and statistical analysis of the experimental results.

Background

The ASTM D3737 method for determining shear strength of horizontally laminated glued-laminated material results in greater design values than does vertically laminated material. Lower design values for vertically laminated members occur because ASTM D3737 assumes every fourth lamination has a through width check or split that limits the strength ratio to one-half. In a four-lamination beam, this results in a total reduction of one-eighth. Based on rational thinking, this approach seems inefficient in light of the ASTM D3737 §4.5, current shear strength research, and laminating concept.

Currently, ASTM D3737 §4.5 states the following about the quality of the laminating member prior to fabrication:

> Lumber shall be free of shakes and splits that make an angle of less than 45° with the wide face of the piece. Pitch pockets shall be limited in size to the area of the largest knot permitted, and pitch streaks shall be limited to one sixth of the width of the lumber.

By this passage, ASTM is requiring a higher quality of material, with respect to splits, shakes, and checks.

Lumber grading rules also specify a maximum allowable split length as a function of the wide dimension of the member. For select structural (SS) and No. 1 material, maximum split length is limited to the width of the wide face. For No. 2 material, the split length is limited to one and a half the member width (WWPA 1991).

The typical probability of no cracks in a piece of wood is given in Table 1, according to the Canadian In-Grade Testing Program. Based on these Canadian data and species, only the 39- by 254-mm (2- by 10-in.) material did not meet or exceed the one in four condition for splits. In addition, the average split length is 0.75 times the width for Douglas Fir, 0.93 the width for Hem-Fir, and 0.62 for Spruce-Pine-Fir (Foschi and others 1989).

Split limits are applied to the lamination material before fabrication. When fabricated in a beam, splits will be restrained from propagation by the surrounding material and cannot develop a full-length split as assumed in the ASTM D3737 criteria. Therefore, quality requirements, split probability, and length characteristics along with split restraint after fabrication seem to make the one in four split assumption conservative.

Since the early 1970s, shear research has moved from the ASTM shear-block to beam-shear experiments. In the first comprehensive study on beam shear, Keenan (1974) indicated that shear strength of Douglas Fir glued-laminated timber beams depended on the sheared area. Also, he observed that the two-beam theory proposed by Newlin and others (1934) is not applicable to glued-laminated timber beams.

Foschi and Barrett (1976, 1977) applied Weibull's theory of rupture to shear strength of wood. The Weibull theory is a statistical theory that predicts the strength of material at a given probability of failure.

Longworth (1977) used a simple-span, Douglas Fir glued-laminated timber beam with symmetrically placed concentrated loads to show experimentally that ASTM shear-block strength is unrepresentative of beam-shear strength, and shear strength varies with beam size, sheared area, or volume.

Quaile and Keenan (1978) used a single-span beam test to investigate maximum shear in rectangular beams of specially designed glued-laminated material. 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

Table I—Probability of no crack in different material (Foschi and others 1989).

	Douglas Fir (%)			Hem-Fir (%)			Spruce-Pine-Fir (%)		
Grade	39 by 254 mm (2 by 10 in.)	39 by 203 mm (2 by 8 in)	39 by 102 mm (2 by 4 in.)	39 by 254 mm (2 by 10 in.)	39 by 203 mm (2 by 8 in.)	39 by 102 mm (2 by 4 in.)	39 by 254 mm (2 by 10 in.)	39 by 203 mm (2 by 8 in.)	39 by 102 mm (2 by 4 in.)
SS	0.82	0.81	0.92	0.94	0.95	0.97	0.90	0.91	0.99
No. 2	0.72	0.76	0.95	0.78	0.86	0.96	0.78	0.87	0.98

lower strength values than is evident in small, rectangular beams. Quaile and Keenan also investigated the effect of growth ring orientation on shear strength. They concluded that radial shear strength values were statistically different than tangential values at a 0.01 level of significance. This indicates that orientation, either flatsawn or quarter sawn, might influence the shear strength of the beam. This orientation effect was also observed in Southern Pine shear-block results (Bendsten and Porter 1978).

Keenan and others (1985) used a special test specimen to determine the shear strength of glued-laminated spruce beams. They concluded that shear strength is not a function of volume but of sheared area.

Rammer and Soltis (1994) experimentally defined the beam-shear strength of Southern Pine and Douglas Fir glued-laminated timber beams. Strength was determined for various-sized beams, 39 by 102 mm (2 by 4 in.) through 127 by 610 mm (5 by 24 in.), by a five-point bending method.

Based on the five-point bending and ASTM D143 shear-blocks results, Soltis and Rammer (1994) empirically derived an equation that related the two findings. Their equation considers the stress riser at the re-entrant comer of the ASTM shear block, different wood species, and the effect of beam size on shear strength. The equation is

$$\tau = \frac{1.9\, C_f\, \tau_{ASTM}}{A^{1/5}} \tag{1}$$

where

τ = beam shear strength, MPa,

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 (MPa), and

A = shear area = area of beam subjected to shear forces (cm^2).

These results were comparable to Longworth's (1977) work on Canadian Douglas Fir glued-laminated timber and Yeh's (1993) work on Douglas Fir glued-laminated timber and laminated veneer lumber beams. Although Equation (1) captures the effect of size on shear strength, the testing program could not easily distinguish between shear area and volume. Therefore, volume could be the significant parameter.

Recently, Janowiak and others (1995) investigated the shear strength of vertically laminated 178- by 229-mm (7- by 9-in.) Red Maple glued-laminated beams. They found that the shear strength of these beams was 10% greater than the values predicted by Equation (1), which is applicable to horizontally laminated beams, but only eight beams of one size were tested.

Glued-laminated material has always boasted of the positive effects of lamination to resist bending forces: improved lumber utilization, more effective transfer of stresses around defects (knots), and reduced strength variability. Positive effects may also be experienced by a deck panel resisting shear forces. A simplistic analogy is used to explain this reasoning.

The major advantage of a glued-laminated beam is its ability to resist bending forces with the cross-sectional moment of inertia (full-composite action). Full-moment resistance may only be accomplished if shear forces are transferred completely through the cross section, from one elemental point to the next point. Inability to transfer shear in one lamination will lead to substantially reduced inertia values, and in the extreme equal, the sum of the individual lamination inertia values (no composite action).

In a beam with full-width horizontal laminations and loaded about its strong axis, shear strength will be governed by the weakest lamination within the middle section of the beam. Delamination and a reduction in the moment of inertia will occur when the shear strength of the weak lamination is exceeded. If the load is maintained or increased, this will lead to failure of the remaining section by tensile rupture or shear.

In a deck panel (same beam as just noted, but loaded about weak axis), the weak lamination will not entirely control the shear strength of the beam. When the weak lamination shear strength is exceeded, the lamination will break and shed load to the surrounding stronger laminations. This load sharing or shedding process will lead to increased performance The larger the deck panel, the smaller the increase of load to the individual surrounding members and the greater the performance.

These failure scenarios are indicative of the two system failure extremes. Essentially the edgewise bending shear failure is an example of a series of chain links; the weak link controls overall system strength. A series system is characterized by a brittle failure and higher variability. The flatwise shear failure is an example of a series of chains acting in parallel to resist the load. The failure of one chain will lead to a load redistribution, which may or may not break the system. Parallel systems are characterized by a more ductile failure.

You could hypothesize that laminated material loaded flatwise could have a shear strength equal or greater than the same material loaded edgewise because of the quality of lamination material, the dependence of shear strength on material orientation, and the reinforcing effects.

Test Program

All previous beam-shear studies concentrated on specimens loaded about its strong axis. In this study, we will test Douglas Fir glued-laminated timber beams to determine both flatwise and edgewise shear strength values and compare the applicability of size–strength relationships for flatwise shear.

A total of 200 matched specimens will be tested: 100 loaded edgewise about the strong axis and 100 loaded flatwise about the weak axis. At the time this paper was written, only the beams loaded edgewise about the strong axis were completely tested along with the smaller-sized beams loaded about the weak axis.

Specimens

To determine the beam-shear strength for various beam sizes, a test matrix covering a range of member sizes was investigated. A standard 1,219-mm (48-in.) wide deck panel cannot be tested due to testing machine limitations. Based on previous shear research, specimens with a shear area greater than 1,935 cm^2 (300 in^2) show slight changes in strength so that the 762-mm- (30-in.-) wide panel could approximate the 1,219-mm- (48-in.-) wide panel (Rammer and Soltis 1994). Tested material consists of Douglas Fir, AITC/ANSI (1995) Combination No. 2 panels. The number and size of specimens tested are listed in Table 2. All material was maintained in a dry but uncontrolled environment until testing. Moisture content ranged between 11% and 13% (on average) at testing.

Beam-Shear Test Setup

A five-point bending test is used to produce a high percentage of beam shear failures. This method has been used successfully to create shear failures by Langley Research Center (Jegly and Williams 1983), Purdue University (Bateman and others 1990), and the Forest Products Laboratory (Rammer and Soltis 1994, Rammer and others 1996). A beam specimen is tested over three supports, and a concentrated load is applied at the middle of each span. This arrangement produces a region of high shear force between the load point and the middle support, making shear failures possible. A schematic of the setup is shown in Figure 1 and corresponding setup dimensions in Table 3. Loading rate is such to cause failure between 5 and 20 minutes, ideally at 10 minutes. Information recorded includes maximum load, type and location of failure, material properties, beam geometry, moisture content (ASTM 1995e), and specific gravity (ASTM 1995c).

Table 2—Size and number of coast Douglas Fir specimens.

Beam size	Specimen length (m)		Number of
(mm)	Flatwise	Edgewise	specimens
130 by 152	1.63	1.70	20
130 by 305	1.63	3.10	20
130 by 419	1.63	4.50	20
130 by 572	1.63	5.89	20
130 by 762	1.63	7.29	20

Shear-Block Tests

Small, clear ASTM D143 shear-block specimens will be cut from the specimens after failure for correlation between previous ASTM D143 shear-block data and beam-shear tests. Shear blocks are cut from material near the failure and along the grain line but free from splits, checks, or knots. Testing will conform to ASTM D143 specifications.

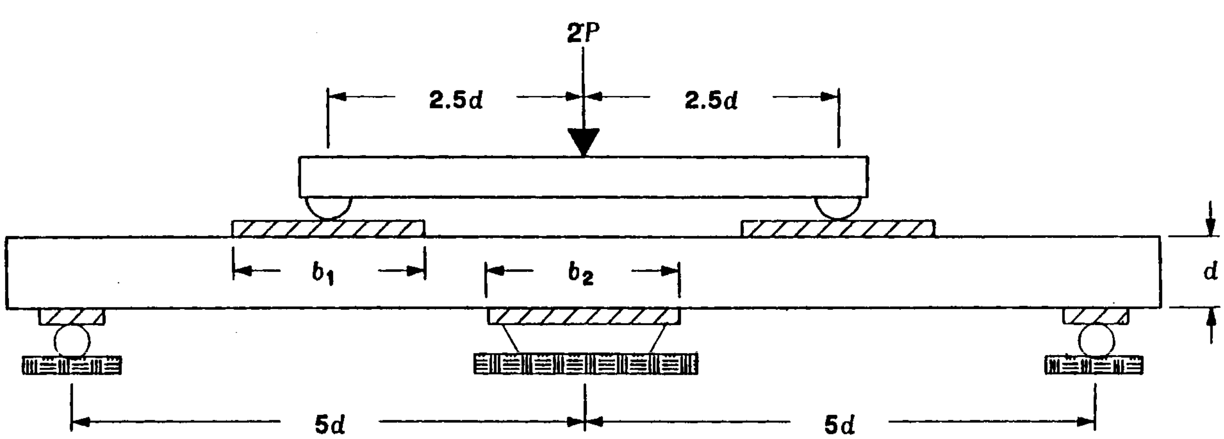

Figure I—Five-point beam-shear test configuration.

Table 3—Beam shear test dimensions.

Orientation of specimen	Beam size (mm)	Span length (m)	
		2.5*d*	5*d*
Flatwise	130 by 152	0.33	0.65
	130 by 305	0.33	0.65
	130 by 419	0.33	0.65
	130 by 572	0.33	0.65
	130 by 762	0.33	0.65
Edgewise	130 by 152	0.38	0.76
	130 by 305	0.76	1.52
	130 by 419	1.05	2.10
	130 by 572	1.43	2.86
	130 by 762	1.90	3.81

Table 4-Beam shear strength results.

Specimen orientation	Beam size (mm)	Number of shear failures	Shear strength (MPa)	COV[a] (%)
Flatwise	130 by 152	9	10.0	8.3
	130 by 305	—	—	—
	130 by 419	—	—	—
	130 by 572	—	—	—
	130 by 762	—	—	—
Edgewise	130 by 152	18	7.71	9.7
	130 by 305	18	6.16	14.9
	130 by 419	19	5.81	12.0
	130 by 572	19	4.82	14.6
	130 by 762	18	4.63	10.2

[a]Coefficient of variation.

Calculations

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

$$\tau = \frac{VQ}{Ib} = \frac{3V}{2bd} \qquad (2)$$

where
τ = Shear stress (Pa)
V = Shear force (N)
b = Width of beam (m)
d = Depth of beam (m)
I = Moment of inertia (m^4)
Q = Statical moment of the area (m^3)

The general equation that relates failure load to shear strength for the five-point test configuration (Fig. 1) is

$$\tau = \frac{33}{32}\frac{P}{bd} \qquad (3)$$

where P is the load applied to one span.

Based on orthotropic analysis procedures (Gerhardt and Liu 1983) of the shear failure region, Equation (3) is within 10% of elastic theory results.

In previous shear strength work, small-sized beams experienced a lower percentage of shear failures. Rammer and others (1996) applied censored statistical techniques to estimate the uncensored mean and coefficient of variation. These statistical techniques will again be applied in this report for analysis of the flatwise beams. In addition, an Analysis of Variance (ANOVA) will be performed to determine if size of beam significantly affects beam-shear strength.

Results

The following summarizes the experimental results for the beam-shear and shear-block testing. Complete details of results and procedures will be published in a future research paper.

Beam-Shear Tests

Five-point bending tests about the strong axis (edgewise) resulted in 92 shear failures out of 100 specimens tested for a 92% shear failure rate. Table 4 lists the average shear strength of only those specimens failing in shear. The edgewise specimen shear strength decreased as the size of the specimen increased, as indicated in Table 4. The coefficient of variation values for Douglas Fir shear strength reported here, 9.7% to 14.9%, are slightly larger than the 8% values observed in glued-laminated Douglas Fir beams (Rammer and Soltis 1994), but are typical for shear strength variance (ASTM 1995d). Note that four beam specimens failed at loads lower than the 3rd percentile values.

Shear-Block Tests

Average shear-block strength and coefficient of variation for edgewise specimens was 7.83 MPa (1,136 lb/in^2). The specific gravity was 0.46 at a moisture content of 10.7%. The shear-block values are similar to the published average strength and variability values (ASTM 1995d).

Discussion

In general, shear failures generated a crack between either load point or middle support. Figure 2 shows the general failure for the 130- by 762-mm (5 1/8- by 30-in.) and 130- by 305-mm (5 1/8- by 12-in.) sizes that represent the upper region of shear strength distribution. Four failures occurred at lower load values than expected. Upon investigation of these four failure

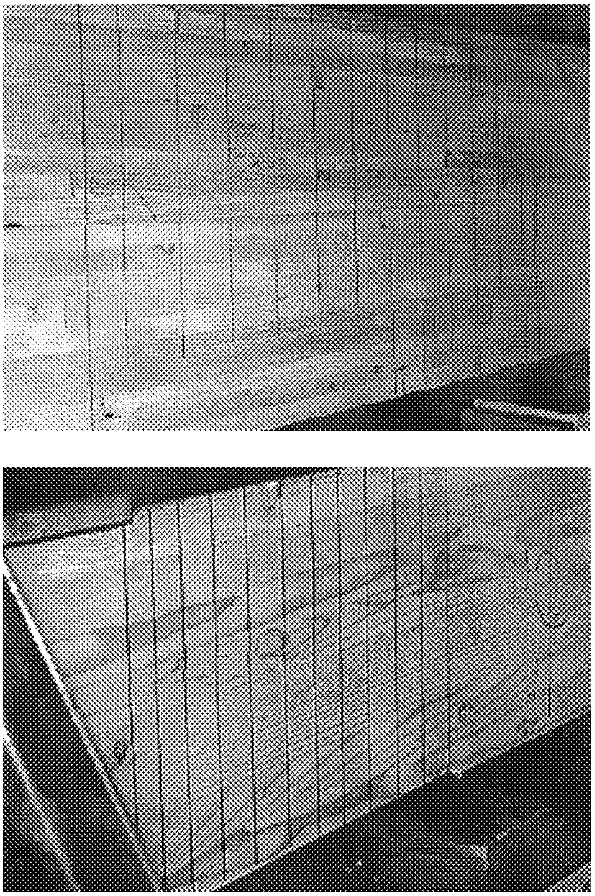

Figure 2—Shear failures in (top) 130- by 762-mm (5 1/8- by 30-in.) and (bottom) 130- by 305-mm beam specimens.

surfaces, one failure occurred at a location of white rot, and two failures were attributed to incomplete bonding of the glueline, (Fig. 3). Incomplete glueline bonding acted like an artificial check or split beam, and the white rot was a plane of extreme shear weakness. These cases illustrate the importance of bonding and wood quality on shear strength. All these failures occurred at loads lower than 3rd percentile levels.

An ANOVA was performed on the edgewise shear strength data, including the four lower load specimens. ANOVA calculations were performed using a general linear model and Tukey’s studentized range test for multiple hypothesis comparison at 0.05 level of confidence (SAS 1988). Based on the ANOVA, three significant groupings resulted. The 130- by 172-mm (5 1/8- by 6-in.) specimens were grouped alone as having the highest shear strength. Next, both the 130- by 305-mm (5 1/8- by 12-in.) and 130- by 419-mm (5 1/8- by 16 1/2-in.) specimens were grouped together.

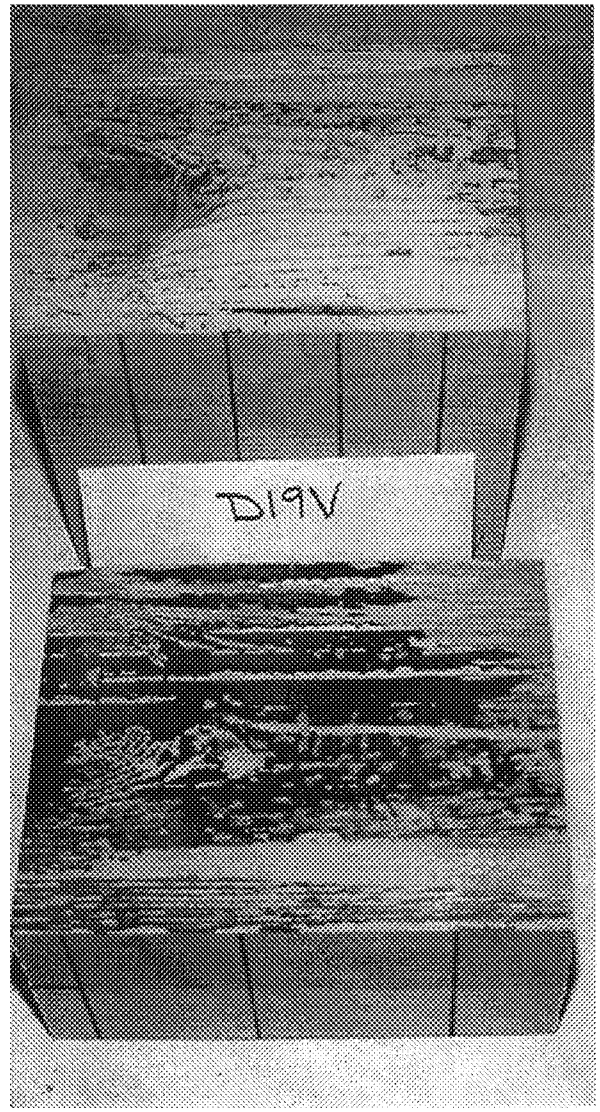

Figure 3—Incomplete glueline bonding in specimen 130 by 305 mm (55 1/8 by 12 in.).

Finally, both the 130- by 572-mm (5 1/8- by 22 1/2-in.) and 130- by 762-mm (5 1/8- by 30-in.) specimens were grouped together, having the lowest shear strength values. The ANOVA indicates that shear strength significantly varied with size, with the smaller-sized specimens having greater strength.

To compare these data with previous glued-laminated shear strength results, the ratio of beam-shear strength to ASTM shear-block strength is compared with shear area and beam volume (Fig. 4). Shear area is taken as the length of beam under high shear forces (distance from load-point to load point = 2.5d, Fig. 1) times the width of the beam, and volume is taken as shear area times beam depth. The edgewise material falls within the variability of previous glued-laminated beam-shear

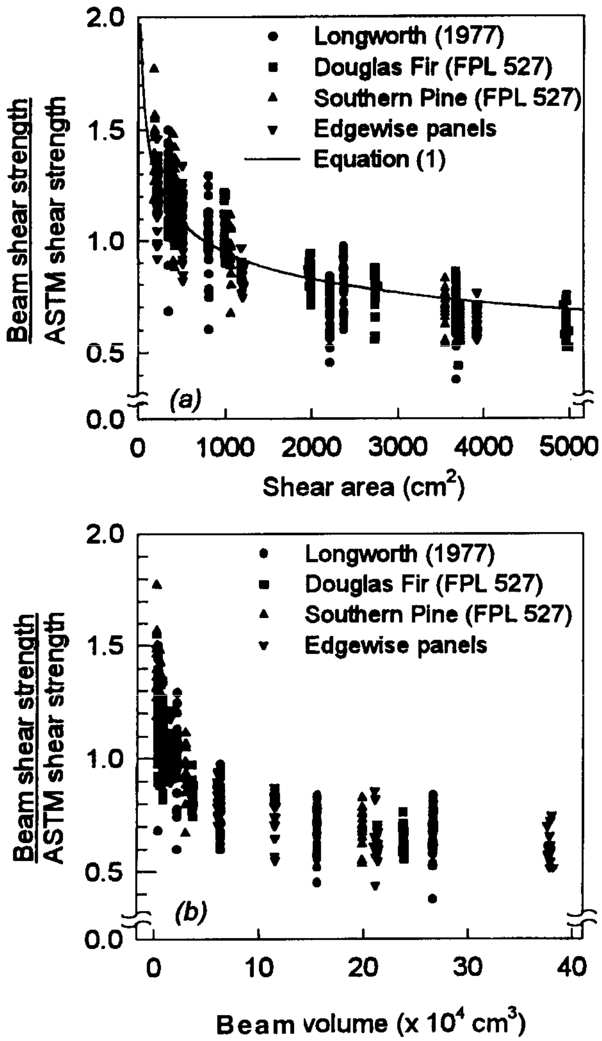

Figure 4—Beam-shear to ASTM shear-block ratio compared with beam size: (top) shear area (bottom) beam volume.

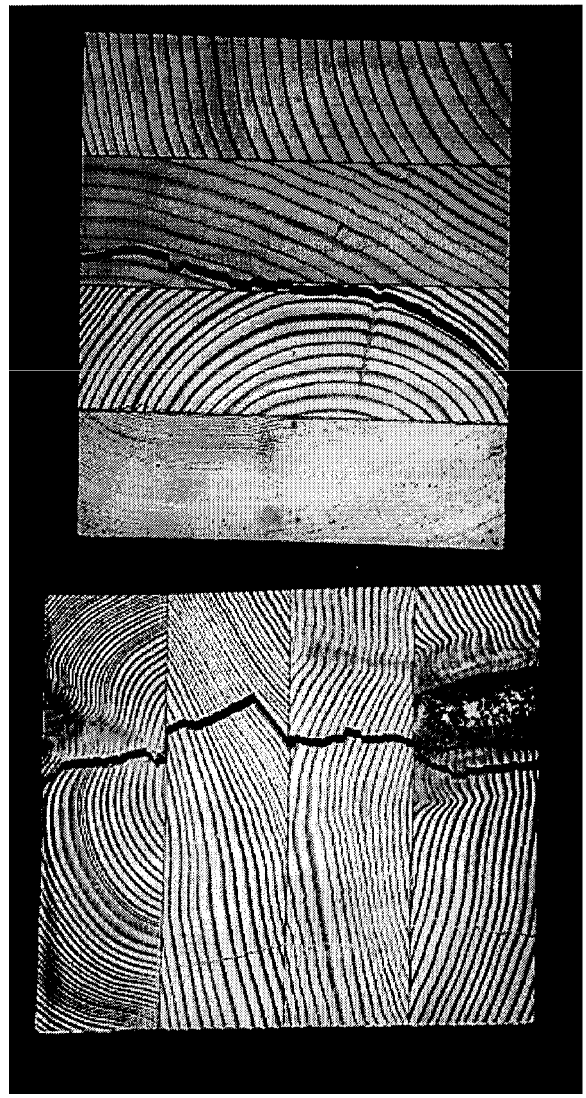

Figure 5—Cross section of failure paths in 130- by 152-mm specimens: (top) edgewise tests (bottom) flatwise.

studies. Results for the flatwise shear are not plotted, because testing was not complete at the time of writing this report.

Flatwise shear strength tests were not completed at the time of report writing, but general observations on smaller beams can be discussed in comparison to the edgewise specimens. In comparing flatwise and edgewise results, it was observed that flatwise values tended to be greater than edgewise values. In fact, a T-test revealed a statistical difference at the 95%. confidence level.

Edgewise shear specimens failed immediately, whereas flatwise failures occurred progressively as indicated by load versus time plots. Applied load increased steadily until a initial shear crack developed and slightly reduced the load. On continued loading, this load reduction was recovered and exceeded, but at a slower loading rate because of the loss of beam stiffness. Subsequently, additional load drops caused by shear or bending failure of the remaining laminations occurred until complete failure of the beam. Therefore, shear failure of the flatwise glued-laminated beams is progressive.

In addition to load-sharing mechanisms, the orientation of the laminations might attribute to increasing the load capacity of the small, flatwise shear results and compared to the edgewise data. All test specimens had a predominant number of flatsawn laminations. This results in edgewise specimens resisting shear forces in the tangential-longitudinal (TL) shear plane, whereas flatwise specimens resisted shear forces in the radial-longitudinal (RL) plane

(Fig. 5). Researchers (Bendsten and Porter 1978, Quaile and Keenan 1978) have indicated that shear strength along these planes is significantly different, with the RL being stronger than the TL. Because edgewise specimens typically failed along the weaker TL plane compared to flatwise specimens that failed across the stronger RL plane, flatwise specimen results could be greater based on material orientation.

In addition to the apparent orientational differences in shear strength, van der Put (1993) formulated that failure criterion is also a function of the material axes. He stated that compression only effects the RL shear strength and has little effect on the TL shear strength. The RL shear strength increases with compression, whereas TL shear strength is constant or slightly decreases. Considering that laminations are predominantly sheared across the TL plane in the edgewise test compared to the flatwise tests and compression has little interaction on the TL shear plane, this would lead to lower shear strength values for the edgewise tests compared to the flatwise tests. Flatwise tests are predominantly shear across the RL plane, which might increase strength with compression.

It should be stated that these comparisons are only based on one specimen size (130 by 152 mm (5 1/8 by 6 in.)); therefore, these observations might not be general for all sizes tested.

Concluding Remarks

An experimental study to investigate the shear strength performance of axial combination Douglas Fir glued-laminated beams and panels is being conducted. Edgewise shear tests are completed and the results fall within the variation of previous glued-laminated shear strength results. An ANOVA indicated a statistical difference in the shear strength with beam size.

Flatwise shear tests are being conducted with only the smaller-sized beams completed. Based on observations of 130- by 152-mm (5 1/8- by 6-in). flatwise tests, a different failure mechanism is presented. Flatwise tests are a progressive failure because each lamination fails in sequence by shear or bending. This progressive failure leads to shear strength results that might be greater than edgewise results. In addition, failure paths in both laminated panels are different.

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