Shear strength of unchecked glued-laminated beams

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Abstract

The allowable stress in shear is derived from shear tests of small clear shear blocks, but the shear strength of shear blocks is much greater than the shear strength of larger beams. In this study, glued-laminated beams were tested to determine shear strength. These specimens were tested in the five-point bending test configuration. Shear blocks were cut from the beam after failure and tested for shear strength. From these tests, a relationship between shear strength and beam size was developed that uses American Society for Testing and Materials (ASTM) shear block strength as a basis. The recommended relationship is based on test results for a number of sizes of Douglas-fir and southern pine unchecked glued-laminated beams. This recommendation also includes the stress concentration factor to account for the effects of the re-entrant comer in the ASTM shear block specimen.

In the early 1900s small clear test specimens were recognized to have higher shear strength than that of larger beam members. Researchers have traced the evolution of allowable shear stress back to about the year 1900 to explain this discrepancy, but no relationship exists (8). The effect of size on shear strength was also studied in Canada and is included in the Canadian standards. Ratios between the small test specimens and large beams were tabulated for various species, This information was the basis for judgmental decisions to define allowable shear stresses. No relationships between the strength of the two sizes were quantified.

A rational attempt to characterize the decreased shear strength with increased beam size began in the 1960s. Huggins and others (12) observed shear strength variation during a Canadian bridge stringer research project. They concluded that the strength variation was a function of the beam depth and the shear span: the shear span is the distance between a support and the nearest applied concentrated load. They stated that the shear strength generally tends to decrease as the ratio of shear span to depth increases. Their data start with high strength values and decrease asymptotically toward a constant value. This asymptotic response is typical of other shear strength studies.

A state-of-the-art report was prepared to summarize all strength-size data and theories (20). This report cites other research reports related to shear strength. Keenan's work with small specimens suggested relating shear strength to the shear area rather than a span-to-depth ratio (14,171. The shear area was defined as the shear span multiplied by beam width. This parameter is easily defined for members with concentrated loads but is undefinable for uniformly distributed loads. Later work by Keenan and others (15) on glued-laminated spruce related shear strength to shear volume rather than shear area. The shear volume was defined as the shear area multiplied by the depth of the beam. Longworth (16) tested shear strength on five sizes of Douglas-fir glued-laminated beams. He also related maximum shear strength to the shear area and volume.

In the 1970s Foschi and Barrett applied the Weibull theory (21) to the tension and shear strength of wood. Barrett (2) applied the theory to the tension perpendicular to grain, and Foschi and Barrett (3,10,11) applied the theory to the shear strength of wood. They related shear strength to an integrated stress volume defined by the Weibull theory.

The Weibull theory is a statistically based theory to predict the strength of material. The attributes of the theory lie in its ability to predict the strength of brittle materials. The following equation is the basis for the theory:

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$$p = 1 - e^{-\int n(\sigma)dV} \quad [1]$$

Probability of fracture p is related to $n(\sigma)$, a positive nondecreasing function, and the volume V. Weibull proposed a simple relationship of

$$n(\sigma) = \left(\frac{\sigma}{\sigma_0}\right)^m \quad [2]$$

which adequately modeled experimental results. After applying the theory to more experimental results, Weibull proposed a new n(o) function as

$$n(\sigma) = \left(\frac{\sigma - \sigma_u}{\sigma_0}\right)^m \quad [3]$$

that would account for the truncation of data by specimen formation process. Equation **[2]** is commonly called a two-parameter Weibull distribution, and Equation **[3]** is a three- parameter Weibull distribution. The three parameters were determined from experimental data.

Each material parameter, m, σ_u, and σ_0, has significance to Weibull's theory, and each will be briefly discussed. The material parameter m characterizes the amount, type, and frequency of flaws in a material. If m is a smaller number, the material tends to be brittle. The material parameter σ_0. is a reference stress level that takes into account the working range of stress. As the variation of the material decreases, the σ_0 values reach a theoretical ultimate strength. The material parameter σ_u, represents the level of stress below which the component or specimen will never fail.

The integral $-\int n(\sigma)dV$ is a function that describes the variation in stress distribution. Because σ is a function of beam depth, span, and width, the parameter cannot be removed from the integral; therefore, the stress distribution from the loading and the material parameter together define the risk of fracture on a given material. Thus, using the Weibull theory for shear strength prediction is difficult because the stress distribution requires numerical integration of the volume integral. Foschi and Barrett (10,11) performed these calculations for various loadings and proposed a design procedure to account for shear stress variation. This approach was adopted in Canadian standards.

Researchers have known since the early 1900s that the results from the American Society for Testing and Materials (ASTM) shear block tests (1) are erroneous because of a stress concentration caused by the re-entrant comer. Many researchers have tried to quantify the concentration value and stress distribution using experimental and analytical methods. The first experimental attempt to determine the effect of the re-entrant corner on stress distribution of the ASTM shear block was by Coker and Coleman (5). These researchers performed photo-elastic experiments on all 1935 timber shear test standards. A 6/ 10 scale prototype of the ASTM shear block was made from the photo-elastic material xylonite. Xylonite is

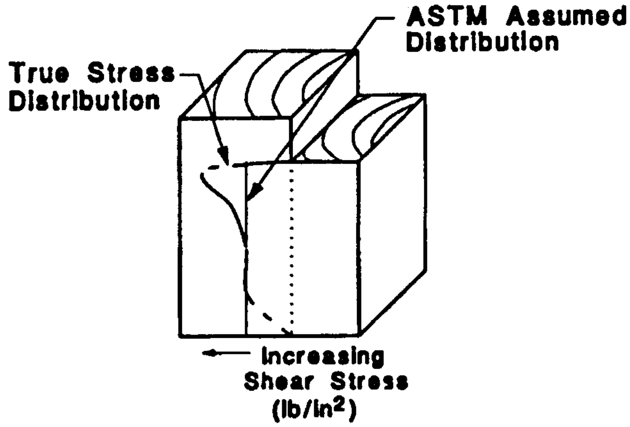

Figure 1 .-Comparison of assumed and true stress distribution in ASTM shear block test (18). (1 psi = 0.0069 MPa.)

considered isotropic: therefore, this material did not truly represent anisotropic wood material. Specimens were loaded only in the elastic response region. From their study, Coker and Coleman concluded that for the ASTM shear block re-entrant comer, maximum shear stress is at least twice the average value.

Yavorsky and Cunningham (22) investigated the strain distribution in glued maple block by brittle coating. Brittle coating indicates strain patterns by cracking on planes perpendicular to tensile components of stress at a relatively consistent strain, as determined by the type of coating used. In areas of pure shear, cracks should develop at 45° to the failure plane. The effect of the re-entrant comer may be estimated by observing the loads that cause cracks near the comer and in the middle of the failure plane.

Radcliffe and Suddarth (18) also investigated stress patterns in the ASTM shear block, modified shear blocks, and a notched beam specimen. The modified shear block was the same as the ASTM standard block but with a horizontal slot cut into the top to reduce the effect of the re-entrant comer. The notched beam was essentially a rectangular beam with a slot cut at the neutral axis. Remaining web width was 25 percent of the original width. Comparison tests were made between specimens and between each specimen configuration.

Radcliffe and Suddarth's measured and assumed stress distributions (18) are shown in Figure 1 for the ASTM standard shear block. The stress concentration factor is approximately twice the assumed value. Distributions for the modified shear block and notched beam indicated some concentration effects, approximately 1.5 times assumed for the modified block and approximately 1.2 times assumed for the notched beam. Radcliffe and Suddarth recommended using the notched beam for determination of shear strength. They did not recommend using the modified shear block, although the values have less error, because the notch caused a large number of tension-perpendicular-to-gram failures. Their results indicate a stress concentration factor of about two for the ASTM shear block.

Cramer and others (7) applied a two-dimensional orthotropic fracture element to the ASTM shear block. Uniform displacement was applied to the loading surface. From the analysis, a stress concentration factor of 2.36 was calculated. This result was higher than experimental results, which was attributed to the localized crushing and splitting near the notch. It was rationalized that these effects would reduce the effect of the re-entrant comer. The finite element analysis found the stress distribution to be the same as that found by experiment.

In summary, the ASTM assumed shear strength is less than the true stress by a factor of about two (18). The finite element analysis indicates a factor larger than two, but these methods did not model effects (crushing and splitting) that might alleviate the stress concentration effect (7). Therefore, we conclude from the available information that an appropriate estimate of the stress concentration factor for the ASTM shear block is two.

TABLE 1. –*Description of test specimens.*

Species	Size	Material layup[b]	No. of specimens
	(in.)[a]		
Douglas-fir	5 by 24	Original 24V-F	20
	4-3/8 by 11-1/2	HQ	33
	3 by 12	Original 24V-F	20
	2-1/2 by 6-1/2	HQ	37
	2-1/2 by 6-1/2	LQ	19
	1-1/2 by 5	HQ	31
	1-1/2 by 5	LQ	32
Southern pine	5 by 22	Original 24V-F	20
	3 by 11	Original 24V-F	20
	2-1/2 by 5-1/2	HQ	37
	2-1/2 by 5-1/2	LQ	39
	1-1/2 by 4	HQ	22

[a] 1 inch = 25.4 mm.
[b] HQ = high-quality laminations: LQ = low-quality laminations.

Experimental procedures

More than 300 glued-laminated beams were tested to determine shear strength. The largest specimens were 5- by 24-inch (0.13- by 0.61-m) Douglas-fir-beams and 5- by 22-inch (0.13- by 0.56-m) southern pine beams. Smaller beams were cut from the lowest-stressed regions (from previous tests) of the larger beams. The largest Douglas-fir specimen was laminated as 24F-V8 as per American National Standards Institute (ANSI) Standard A190.1, and the largest southern pine specimen was 24F-V5 as per the same standard. The smaller beams were cut from various

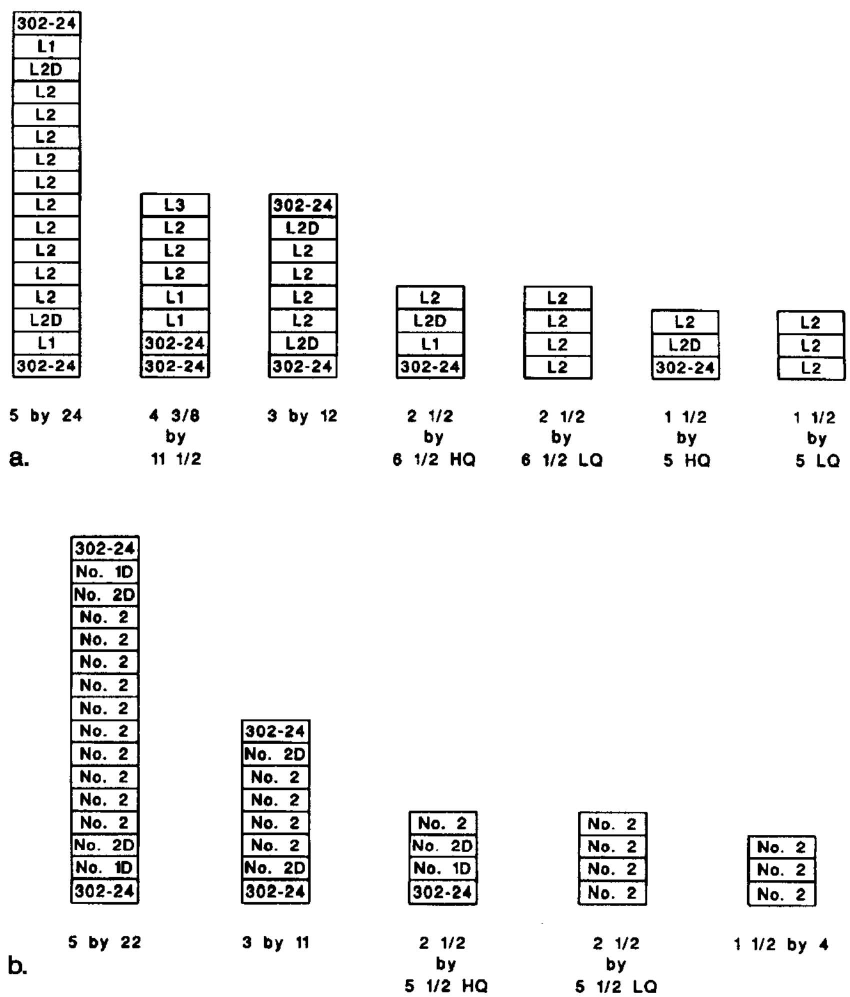

Figure 2.- Beam lamination: a) Douglas-fir; b) southern pine. HQ = high quality; LQ = low quality. (Sizes in inches, 1 in. = 25.4 mm).

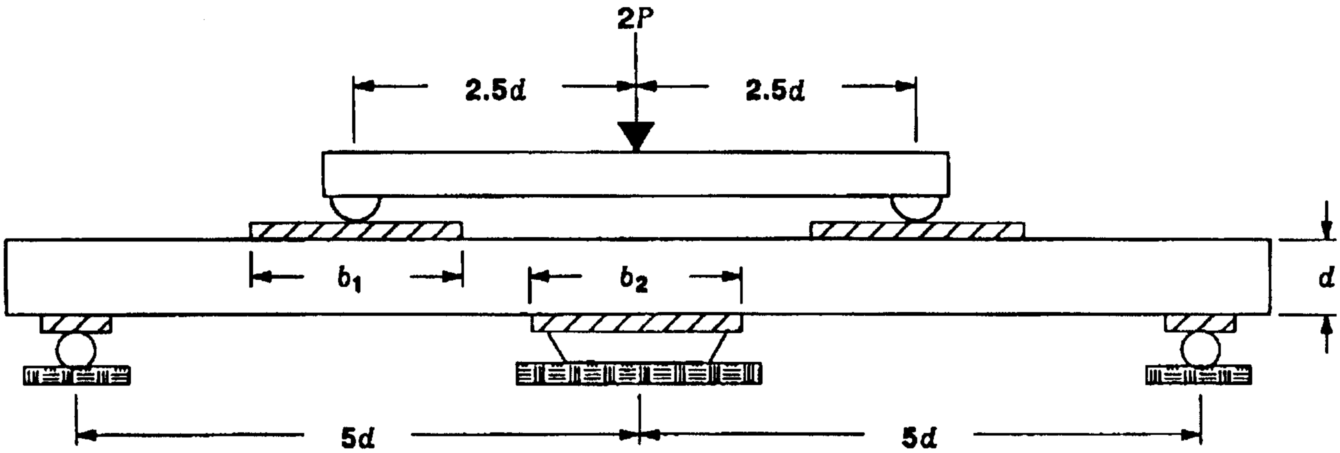

Figure 3. - Five-point bending test configuration. P = load applied; d = beam depth; b = length of bearing.

TABLE 2. -Beam shear strength.

Species	Size	Shear strength[a]	No. of shear failures[b]
	(in.)	(psi)	
Douglas-fir	5 by 24	770 (8.7)	20 (20)
	4-3/8 by 11-1/2	940 (10.1)	31 (33)
	3 by 12	1,000 (6.7)	20 (20)
	2-1/2 by 6-1/2 HQ	1.280 (10.8)	32 (37)
	2-1/2 by 6-1/2 LQ	1,240 (7.7)	14 (19)
	1-1/2 by 5 HQ	1.440 (8.3)	28 (31)
	1-1/2 by 5 LQ	1.450 (9.7)	26 (32)
Southern pine	5 by 22	970 (10.4)	20 (20)
	3 by 11	1,310 (10.9)	20 (20)
	2-1/2 by 5-1/2 HQ	1.630 (11.4)	28 (37)
	2-1/2 by 5-1/2 LQ	1.630 (9.0)	22 (39)
	1-1/2 by 4	1.970 (9.1)	12 (22)

[a] Values in parentheses are coefficients of variation. One psi = 0.0069 MPa.
[b] Values in parentheses are the total number of tests.

locations in the larger beams. Thus, some had high quality (HQ) material for laminations, whereas others had low quality (LQ) material. The nominal sizes, material layups, and number of specimens are given in Table 1 and Figure 2. A full description of test procedures and results is given in a companion report (19).

A five-point bending test configuration was used to consistently produce shear failure (Fig. 3). The configuration consisted of three supports and two concentrated loading points located equidistant from the middle support. The specimen length was 10 times the depth of the member, with each individual span equal to 5 times the depth. Concentrated loads were applied at midspan of each individual span. This same test configuration has been used for composites (4) and composite laminates (13). The loading was deformation controlled until the specimen failed. Failures occurred 5 to 15 minutes after loading started. Once the beam specimen failed in shear, an ASTM shear block (Fig. 1) was cut from the beam and tested according to ASTM D 143 (1). In southern pine, shear blocks were cut randomly from beams from the end of the failed specimen at the same height as the shear crack. In Douglas-fir, the shear blocks were cut at the same height as the shear crack and within the same member but were horizontally displaced.

Specimens were stored and tested in a noncontrolled environment. Moisture contents were measured after each test; specimens ranged from 9 to 13 percent. Shear block specimens were conditioned to about 11 percent moisture content prior to testing.

Results

Three failure modes were predominant during testing: excessive compression perpendicular to gram near the reactions, bending at the middle support, and shear mid-depth in the beam and between the load and center support. Only shear failures are included in the results.

Loads are converted to shear strength using elementary mechanics and assuming glued-laminated beams are homogeneous cross sections:

$$V = \frac{11P}{16} \quad [4]$$

$$\tau = \frac{VQ}{Ib} \quad [5]$$

where:

V = shear force

P = load applied
τ = shear stress
Q = static moment of the area
I = moment of inertia

The maximum strength (τ_{max}) is calculated by substituting the maximum shear force and considering the rectangular cross section. After inserting these factors, the equation for maximum strength is:

$$\tau_{max} = \frac{33P}{32bd} \quad [6]$$

where:
b = width of the beam
d = depth of the beam

Elementary parabolic shear stress distribution is not valid within the region of the supports (6). Our shear failures were far enough from the supports that we used the elementary mechanics approach. Table 2 shows the average shear strength values, coefficients of variation, and numbers of shear failures compared to the total number of tests for the Douglas-fir and southern pine specimens. Most failures occurred in the wood with some failures occurring in the gluelines. Research in progress indicates little effect of combined shear and bending stresses. Thus, this effect is assumed negligible for this study.

The ASTM D 143 shear block tests resulted in an average maximum shear stress of 1,146 psi (7.9 MPa) for Douglas-fir and 1,382 psi (9.5 MPa) for southern pine with coefficients of variation of 19.9 and 16.0 percent, respectively. These species correspond to those most commonly used in the areas where the beams were fabricated. The Forest Products Laboratory's Wood Handbook (9) shear strength values are 1,130 psi (7.8 MPa) for coast Douglas-fir and 1,390 psi (9.6 MPa) for loblolly pine. Table 3 summarizes the shear strengths and coefficients of variation for each beam size.

TABLE 3. –*ASTM shear block strength*

Species	Size of specimen source	Average ASTM shear strength
	(in.)	(psi)
Douglas-fir	5 by 24	950 (27.7)
	4-3/8 by 11-1/2	1,530 (17.7)
	3 by 12	1.110 (16.4)
	2-1/2 by 6-1/2 HQ	1.310 (9.5)
	2-1/2 by 8-1/2 LQ	1.230 (20.3)
	1-1/2 by 5 HQ	1,230 (11.2)
	1-1/2 by 5 LQ	1,120 (11.6)
Southern pine	5 by 22	1,340 (14.4)
	3 by 11	1,420 (12.9)
	2-1/2 by 5-1/2 HQ	1.390 (16.8)
	2-1/2 by 5-1/2 LQ	1.360 (18.5)
	1-1/2 by 4	1.380 (10.8)

[a] Values in parentheses are coefficients of variation.

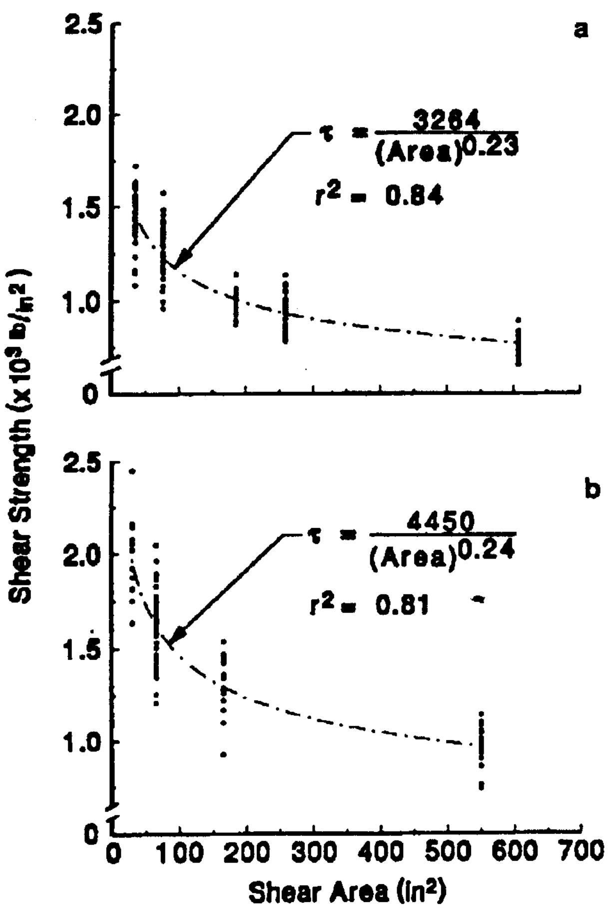

Figure 4. - Shear strength and shear area regression for a) Douglas-fir beams and b) southern pine beams. (1 psi = 0.0069 MPa, 1 in.^{2} = 0.65 mm^2.)

Discussion

As discussed previously, three parameters have been used relative to shear strength: shear span-to-depth ratio, shear area, and integrated stress volume for specific loading and material parameters. The shear span-to-depth ratio is dependent on the location of concentrated loads and is hard to present in code applications. The stress volume requires numerical integration and is also difficult to apply. Therefore, we decided on a shear area approach. We defined the

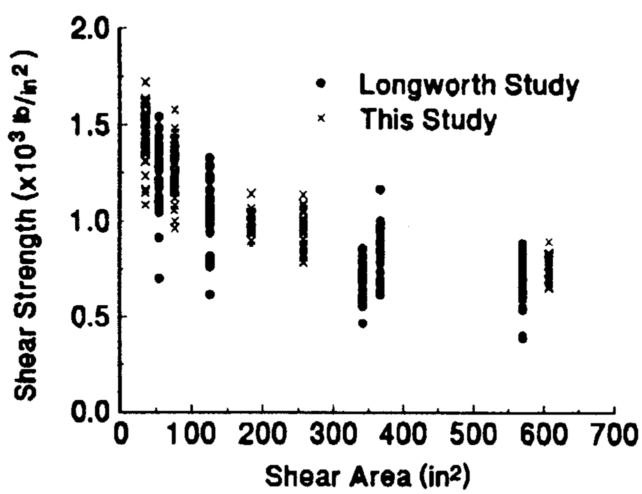

Figure 5. - Comparison of Douglas-fir data with those of Longworth (16).

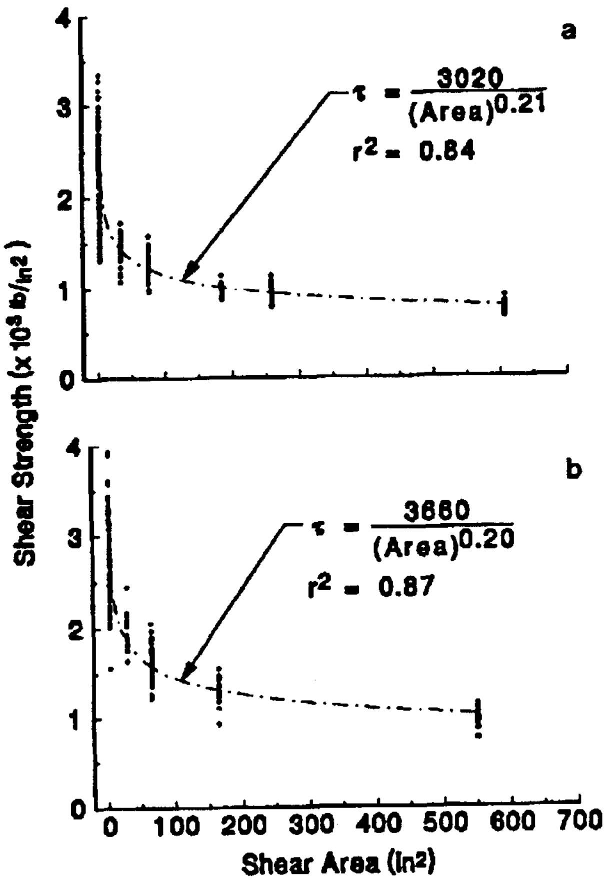

Figure 6. - Shear strength and shear area regression for a) Douglas-fir beams and block shear specimens and b) southern pine beams and block shear specimens.

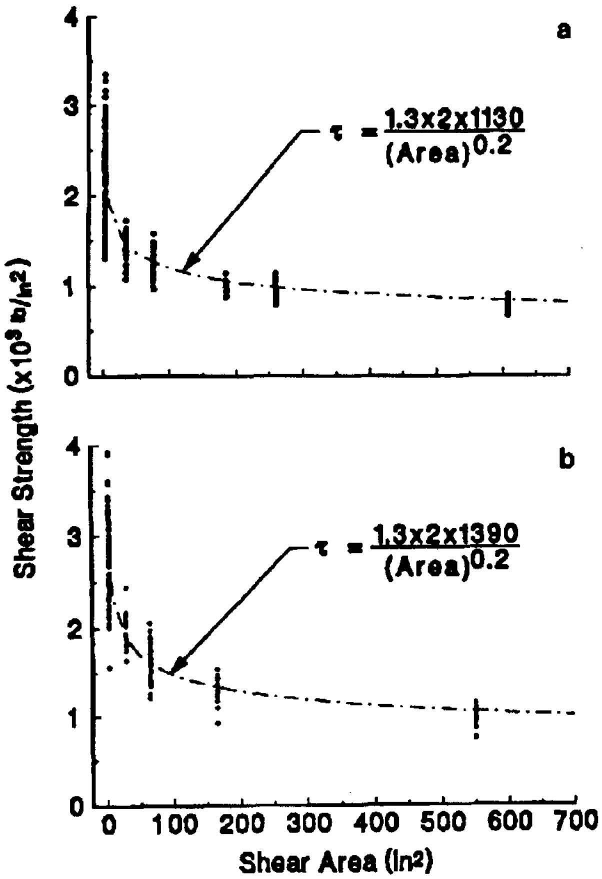

Figure 7. - Equation **[11]** plotted for the a) Douglas-fir shear data and b) southern pine shear data.

shear area as the width of the beam multiplied by the total length of the beam under a shear force action in one span. As in the case of the test configuration presented previously, the shear area is $5bd$. For a beam under third-point loading, the shear area would be $0.67LB$ where L is the span length. Keenan and others (15) defined shear area as the shear span times the beam width. The shear span was defined as the length of the beam under positive shear. We defined shear area as the beam length under both positive and negative shear. Thus, our definition would result in a shear area two times that of Keenan's for most cases encountered in engineering design. Our definition is probably easier to apply because in most cases the shear span equals the length of the beam.

Figure 4 shows shear area compared to shear strength for the Douglas-fir and southern pine beam specimens. For both species, the shear strength decreased as the shear area increased. Shear strength decreased quickly for small shear areas and asymptotically for large shear areas. The shear strength of southern pine was consistently greater than that of Douglas-fir. A power curve was regressed independently through the Douglas-fir and southern pine shear strength values (Fig. 4). We also compared the Douglas-fir data with those of Longworth (16) (Fig. 5). A visual comparison indicated good agreement between the data sets.

For the Douglas-fir glued-laminated beams, the regression analysis resulted in:

$$\tau = \frac{3{,}264}{A_s^{0.23}} \qquad [7]$$

where:

A_s = the shear area

The regression coefficient (r^2) was 0.84. For the southern pine glued-laminated beams, the regression analysis resulted in:

$$\tau = \frac{4{,}450}{A_s^{0.24}} \qquad [8]$$

The regression coefficient was 0.81. Both equations have nearly the same exponent, which is good evidence that the shapes of the curves are independent of species. Also, both curves have regression coefficients that indicate a high degree of correlation between the shear area parameter and maximum shear strength. The only difference is the constant in the numerator of the equation, which reflects the difference in shear strength of the species.

The data from the beam shear tests were combined with the data from the shear block tests (Fig. 6). The shear block data were adjusted by a stress concentration factor of two. (Refer to the previous discussion on stress concentration factors for the ASTM shear block.) The shear area is 4 in.^{2} (22.6 mm^2) for the shear block.

Performing the regression analysis with the inclusion of the shear block data resulted in an equation for the Douglas-fir data:

$$\tau = \frac{3{,}020}{A_s^{0.21}} \qquad [9]$$

The regression coefficient was 0.87. For the southern pine data, the regression is represented by the following:

$$\tau = \frac{3{,}660}{A_s^{0.20}} \qquad [10]$$

The regression coefficient was 0.87. The shear block data set was larger than that for the beam shear data, which weights the curve. We compared the regression curve for the overall data with a regression curve for the mean values at each size. The mean value regression does not have the weight effect of the larger shear block data set. The two regressions were nearly identical: thus, we neglected the weighting effect.

The constant in the previous regression equations represents the shear strength corresponding to a shear area of 1 in.^{2} (0.6 mm^2). The shear area of the ASTM block is 4 in.^{2} (2.6 mm^2). Thus, these equations could be rewritten in terms of the shear block strength rather than the strength corresponding to 1 in.^{2} by adjusting by a factor of 1.3. The exponents in Equations **[9]** and **[10]** are both approximately equal to 1/5. Thus, an approximate recommended equation for both species is:

$$\tau = \frac{1.3 C_f \tau_{ASTM}}{A_s^{1/5}} \qquad [11]$$

where:

τ = beam shear strength (psi)

C_f = 2, the stress concentration factor to adjust the ASTM shear block assumed to be true stress distribution

τ_{ASTM} = ASTM D 143 published shear block values (psi)

A_s = shear area (area of beam subjected to shear forces (psi))

Equation **[11]** 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 southem pine data in Figure 7, which shows that Equation **[11]** approximates the data quite well.

Conclusions

It has long been known that ASTM shear block strength is greater than beam shear strength. Researchers have also observed that beam shear strength decreases as beam size increases. The results of this study reaffirm these observations for Douglas-fir and southern pine unchecked glued-laminated beams. Equation **[11]** is recommended to relate beam shear strength to beam size using ASTM shear block strength as a basis. Equation **[11]** includes a stress concentration factor to account for the effects of the re-entrant corner in the ASTM shear block specimen. The proposed equation has two advantages: a) the equation takes the species into account by using the appropriate ASTM shear block value and b) the shear area parameter is easy to calculate and can be conservatively estimated as the entire length of the beam multiplied by the width.

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