# STRENGTH AND STIFFNESS OF LAMINATED DOUGLAS-FIR BLOCKS IN PERPENDICULAR-TO-GLUELINE TENSION

*Selwyn P. Fox*

Research Scientist, Department of the Environment, Canadian Forestry Service,
Western Forest Products Laboratory, Vancouver, B. C. V6T 1X2

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ABSTRACT

Blocks of commercial glued-laminated Douglas-fir were tested to failure in tension perpendicular-to-gluelines. The results of a two-level factorial experiment indicated that blocks 22 inches in length were weaker in strength than blocks 7 inches in length and that blocks of 5- × 5-inch cross section were weaker than blocks 3 × 3 inches. Additional longer specimens were tested to provide better estimates of their ultimate strength.

The average modulus of elasticity of 3- × 3-inch blocks was about 70,000 psi. Undetected ring shake caused early failure of some specimens, indicating that this natural characteristic might be responsible for unexplained failures of beams in service.

According to Weibull, specimen size affects material strength. To confirm application of his work to these tests, additional blocks of three other configurations were tested for strength. An assumption of a log–log relation as hypothesized by Weibull is acceptable. Test results of other researchers found from similar specimens show no noticeable deviation from the relationship.

*Additional keywords: Pseudotsuga menziesii,* glued-laminated beams, tension tests, pitched-tapered beams, size effects.

## INTRODUCTION

One structural member that is popular for building construction, where a sloping roof combined with maximum interior clearance is desirable, is the pitched-tapered glued-laminated beam typified by Fig. 1. For unknown reasons, some beams of this type fail in service but not catastrophically. The form of a typical failure is a separation near the neutral axis in the curved portion of a beam (Hanrahan 1966). Studies of the stress distribution induced in pitched-tapered beams by live loads have been described by Foschi and Fox (1970); Foschi (1971); Fox (1970); and most recently by Gopu et al. (1972). They have shown by theory and experiment that the radial stresses in the central curved-tapered portion of these beams are underestimated when the Wilson formula (1939) for curved beams of constant cross section is used.

A new analysis method (Foschi 1970) does not explain all of the difference between failure load and estimated capacity based on the tension-perpendicular-to-grain stress for clear straight-grained Douglas-fir as published by Kennedy (1965). Thus, an assessment of the strength of dry Douglas-fir [*Pseudotsuga menziesii* (Mirb.) Franco] in tension perpendicular-to-grain (glueline) was considered necessary to understand the problem of in-service failures.

Various types of specimens have been described by Markwardt and Youngquist (1956), who stated that wide variation in strength has been found between different specimen types. A recent example is the mean strength difference reported by Schniewind and Lyon (1973) for two sizes of Douglas-fir unglued specimens. A rectangular glued-laminated block of uniform cross section (Fig. 2) has been utilized by several researchers (Thut 1970; Madsen 1972; Peterson[1]; Moody[2]). Under an axial force that is perpendicular-to-the-gluelines, a block is subjected to a uniform tensile

[1] Peterson, John. 1973. Oregon State U., Corvallis. Personal communication.

[2] Moody, R. C. 1974. U.S. Forest Products Laboratory, Madison, Wis. Personal communication.

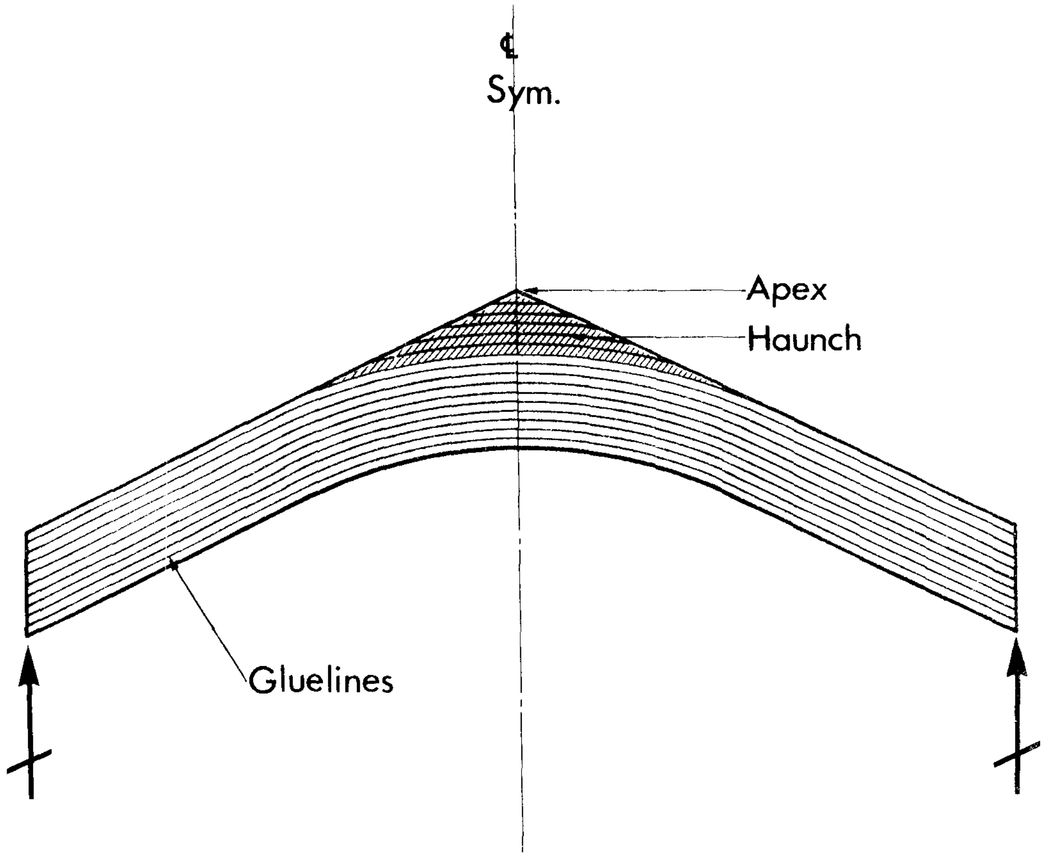


FIG. 1. Pitched-tapered glued-laminated beam.

stress along most of its length. Exploratory tensile tests by Thut (1970) of 22 glued-laminated Douglas-fir blocks, 4½ × 4½ × 13 inches, showed an average strength of 128 psi and a standard deviation of 42 psi. One-minute load-duration strengths found by Madsen (1972) for 15 blocks, 5 × 5 × 24⅝ inches, cut from three glued-laminated beams, averaged 141 psi with a standard deviation of 42 psi. Eleven other blocks of the same dimensions cut from three other special beams made from clear boards averaged 191 psi with a standard deviation of 40 psi. Another axial tension-perpendicular-to-glueline study of 44 laminated 2½- × 2½- × 9-inch Douglas-fir blocks is unpublished (Peterson[1]) and yielded an average strength of 198 psi with a standard deviation of 68 psi. Yet another unpublished study (Moody[2]) of 25 clear blocks 1 × 1 × 1½ inches cut from five failed curved-laminated Douglas-fir model beams yielded an average strength of 597 psi with a standard deviation of 171 psi. These examples support the observations of Markwardt and Youngquist (1956) and are summarized in Table 3.

These values contrast sharply with those derived from tension-perpendicular-to-grain tests of clear, straight-grained Douglas-fir by national laboratories. The Canadian average is 443 psi for 374 specimens with a standard deviation of 154 psi. Comparable U.S. tests yielded an average strength of 340 psi (U.S.D.A. 1955), indicating a wide variation between means within a species despite practically identical specific gravities (0.49 volume air-dry and 0.48 volume at 12% MC, respectively).

The objective of this report is to relate the

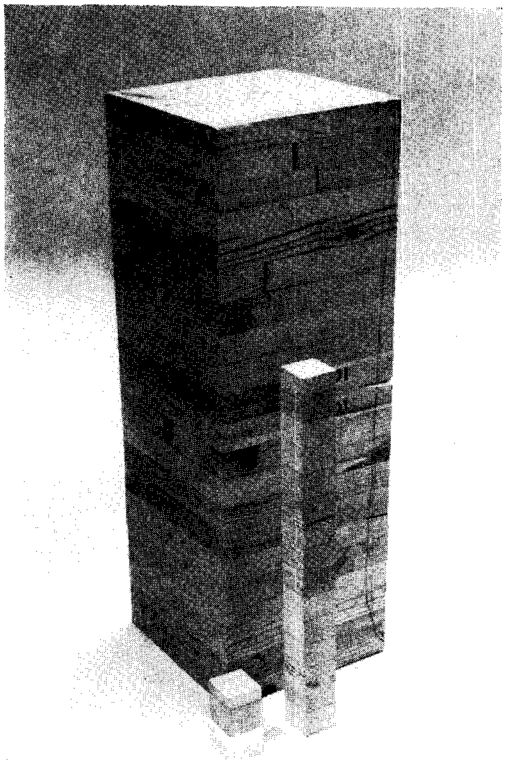

FIG. 2. Three configurations of blocks.

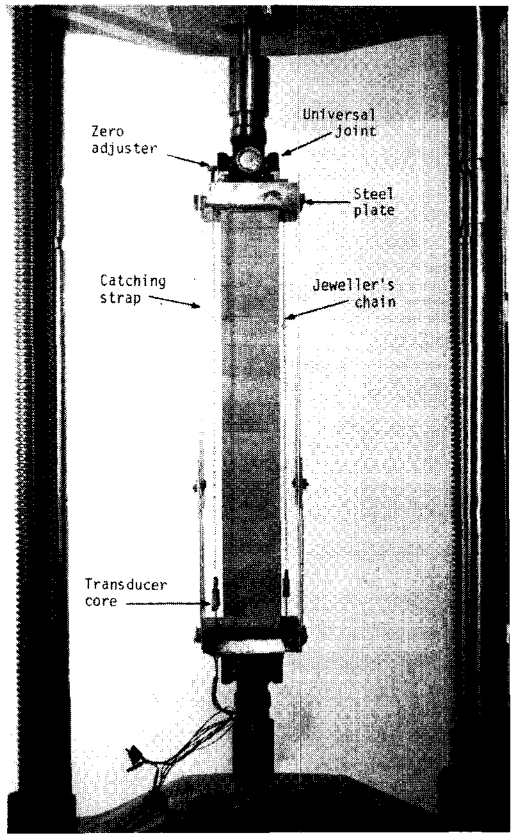


FIG. 3. Test setup for 3- × 3-inch specimens.

different average tension-perpendicular-to-glueline strengths of randomly selected Douglas-fir glued-laminated blocks to different configurations and volumes (Fig. 2). These results are then reviewed in relation to the strength in radial tension of some pitched-tapered beams and current allowable unit stresses.

## METHOD

### *Factorial experiment*

Four factors were included in an initial study—specimen length (perpendicular-to-gluelines), cross-sectional area (parallel-to-glueline plane), moisture content, and testing speed. Two levels of each factor were studied providing a $2^4$ factorial experiment. Seven replications were provided for each group. Thus, 112 specimens were selected randomly from 112 end-trims of laminated beams. The factors and their levels are shown in Table 1.

### *Further sampling*

Additional specimens of the longer length were prepared in anticipation of assignment of working stresses. Forty-seven additional 3- × 3- × 22-inch and 38 additional 5- × 5- × 22-inch specimens were tested.

Since the above tests indicated that mean strength decreased with volume, three other configurations were prepared—30 specimens 2 × 2 × 4 inches, 30 specimens 2 × 2 × 20 inches, and 22 specimens 10¾ × 10¾ × 34 inches.

### *Materials*

For the initial study, arrangements were made with four laminating factories for twice-weekly collection of end-trim pieces from their products. Each end-trim pro-

TABLE 1. *Factor-level means*

| Factor | Levels 1 | Levels 2 | Mean strengths, psi 1 | Mean strengths, psi 2 |
|---|---|---|---|---|
| Length | 7 inches | 22 inches | 181 | 133 |
| Cross section | 3 x 3 inches | 5 x 5 inches | 169 | 145 |
| Moisture content | 6% | 18% | 158 | 156 |
| Testing speed | 0.02 inches/min | 0.10 inches/min | 151 | 163 |

duced one unique specimen containing natural wood characteristics, such as knots, wane, slope of grain, ring shake, and checking. These end-trims were cut to cross-sectional sizes of 3½ × 3½ inches or 5½ × 5½ inches before being placed in either a 6% equilibrium moisture content (EMC) or 18% EMC chamber. All blocks consisted of laminations of 2-inch nominal thickness kiln-dried to less than 16% MC. Blocks to be conditioned to 6% MC were treated with two coats of an epoxy finish applied to the end-grain to prevent checking of those surfaces. After conditioning for at least six weeks, all potential specimen blocks were inspected. Since the effect on strength of checks would be difficult to evaluate, any blocks that exhibited visible checking were excluded unless a check was aligned with a block's length—i.e., parallel to the force to be applied to block ends. Specimens were cut to final size by bandsaw. Heavy steel plates were attached to each block with an epoxy adhesive mixture. An alignment pin was used to center specimen ends on the plates. Specimens were then conditioned for 20 h to allow the adhesive to cure under contact pressure.

A similar technique was used for subsequent specimens. The MC of these blocks varied from 7 to 23%. The plates applied to the largest blocks were held by screws. These largest blocks came from one factory but were from different beams. There was an unmeasured higher proportion of clear wood in the 2- × 2-inch sets.

TABLE 2. *Analysis of variance for the factorial investigation*

| Source | Degrees of freedom | Sum of squares | Mean square | F ratio |
|---|---|---|---|---|
| Length | 1 | 65,138 | 65,138 | 28.02[a] |
| Cross-section size | 1 | 16,734 | 16,734 | 7.20[b] |
| Moisture content | 1 | 62 | 62 | 0.03 |
| Testing speed | 1 | 3,623 | 3,623 | 1.56 |
| Interactions | 11 | 15,392 | 1,399 | 0.60[c] |
| Error | 96 | 223,160 | 2,325 | |
| Total | 111 | 324,110 | | |

[a] Probability level approximately 0.0001%, i.e., there is about one chance in a million that this value would be attained if there were, in fact, no difference attributable to length.

[b] Probability level approximately 0.86%.

[c] None of the interactions were significant. They have been pooled for convenience in presentation.

### *Testing*

Universal joints were used to reduce eccentricity inherent in testing apparatus, as shown in Fig. 3. The use of a universal joint as close as possible to a specimen has been demonstrated to reduce test-result scatter (Penny and Leckie 1968).

Estimates for modulus of elasticity perpendicular-to-grain of wood are 1/10 to 1/20 of the longitudinal modulus (U.S.D.A. 1955). To provide some insight about this property, the 3- × 3-inch specimens were instrumented with displacement transducers in pairs to nullify bending effects. The gauge length was the distance between the steel plates at the centroidal axis of a specimen, minus an average epoxy-glueline thickness. The combined signal of these two transducers was plotted by an x-y recorder against the load signal of the testing machine until failure occurred. Testing

TABLE 3. *Strength and stiffness summary for tension perpendicular*

| Set no. | Set dimensions, inches | Mean volume, inches$^3$ | Strength, psi | | | | | | Modulus of elasticity, psi | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | n | Min. | Ave. | Max. | Std. dev. | Mean s.g.[(b)] | n | Min. | Ave. | Max. | Std. dev. |
| 1 | 3 x 3 x 7 | 65.8 | 29[(a)] | 90 | 191 | 278 | 49.1 | 0.50 | 26 | 26200 | 75547 | 143500 | 28244 |
| 2 | 5 x 5 x 7 | 172. | 28 | 80 | 172 | 309 | 49.0 | 0.50 | - | | | | |
| 3 | 3 x 3 x 22 | 187. | 75 | 31 | 152 | 319 | 59.4 | 0.50 | 64 | 35800 | 68180 | 154500 | 17352 |
| 4 | 5 x 5 x 22 | 506. | 66 | 57 | 126 | 198 | 37.4 | 0.50 | - | | | | |
| 5 | 2 x 2 x 4 | 15.8 | 30 | 131 | 356 | 527 | 100. | 0.53 | - | | | | |
| 6 | 2 x 2 x 20 | 80.1 | 30 | 65 | 180 | 319 | 71.1 | 0.48 | - | | | | |
| 7 | 10 3/4 x 10 3/4 x 34 | 3650. | 22 | 55 | 100 | 147 | 19.3 | 0.46 | - | | | | |
| 8 | Thut, 1970 | 263. | 22 | 83 | 128 | 228 | 41.9 | - | - | | | | |
| 9 | Madsen, 1972 | | | | | | | | | | | | |
| | "commercial" | 616. | 15 | 64 | 141 | 209 | 42.4 | - | 14 | | 103000 | | 17750 |
| | "clear" | 616. | 11 | 108 | 191 | 242 | 39.8 | - | 11 | | 85480 | | 26530 |
| 10 | Peterson[1] | 56.2 | 44 | 38 | 198 | 288 | 67.5 | - | - | | | | |
| 11 | Moody[2] | 1.5 | 25 | 65 | 597 | 920 | 171. | 0.48 | - | | | | |

(a) An additional specimen was tested after the factorial experiment.

(b) Oven-dry volume basis.

machine speeds of 0.02 and 0.10 inches/min were used for the factorial study, but only the latter speed was used for subsequent tests.

After failure of a specimen and disassembly of the apparatus, a sample that included the fractured surface was cut for MC and specific gravity (oven-dry volume basis) measurements.

## RESULTS

### *Factorial experiment*

Table 1 shows the mean values obtained and Table 2 shows the analysis of variance results for the $2^4$ factorial experiment. There was no significant difference in strength between 56 specimens at a nominal 6% MC and 56 specimens at a nominal 18% MC. Similarly, the two levels of testing speed did not result in significantly different strengths. The larger specimens of level 2 were significantly weaker than those of level 1.

### *Additional longer block tests*

Since there was no apparent difference between levels of MC and testing speed, the test values of the factorial experiment were pooled according to cross section and length and then combined with the strengths of the additional longer specimens tested. Sets 1 to 4 of Table 3 summarize the strengths found for these 198 specimens.

Test results collected from three other configurations—2 × 2 × 4 inches, 2 × 2 × 20 inches, and 10¾ × 10¾ × 34 inches—are also summarized in Table 3 as sets 5 to 7. It is apparent that average block strength decreases with an increase of specimen volume.

Table 3 lists the calculated moduli of elasticity for tension perpendicular-to-glueline (grain). When the minimum and max-

TABLE 4. *Residuals for regression line*

| Experimental means: Strength, psi | Volume, inches$^3$ | Estimated average strength, psi | Residual |
|---|---|---|---|
| Using set nos. 1-7 (Table 3) | | | |
| 356 | 15.8 | 287 | -69 |
| 191 | 65.8 | 207 | +16 |
| 180 | 80.1 | 198 | +18 |
| 172 | 172 | 166 | - 6 |
| 152 | 187 | 163 | +11 |
| 126 | 506 | 129 | + 3 |
| 100 | 3650 | 82 | -18 |
| Using set nos. 8-11 (Table 3) | | | |
| 198 | 56.2 | 213 | +15 |
| 141 | 616 | 122 | -19 |
| 128 | 263 | 149 | +21 |
| 597 | 1.5 | 490 | -107 |

imum values are compared with an average parallel-to-grain modulus of 1.93 million psi for Douglas-fir (C.S.A. 1970), ratios of 1:74 to 1:13 are obtained, which is wider in the low direction than the 1:20 to 1:10 ratios currently recommended (U.S.D.A. 1955). The ratio of one mean to the other is 1:27, which is also outside this recommended range. Since only two lengths of 3- × 3-inch blocks were tested, no consideration was given to effect of volume on modulus of elasticity.

## ANALYSIS

Using sets 1 to 7 of Table 3, a plot of mean strength versus specimen volume indicated a nonlinear relationship. As hypothesized by Weibull (1939), better linearity was obtained by plotting logarithm of mean strength against logarithm of volume. This regression line is $\log_{10}$ (average strength) $= 2.73 - 0.230 \log_{10}$ (specimen volume). Table 4 contains the residuals for the test averages weighted by n, the number of observations.

Sets 8 to 11 of Table 3, representing the tests of Thut, Peterson, Moody and the "commercial" values of Madsen, fit closely to the log–log regression line. Madsen's "clear" material was rejected, since it is not typical for commercially produced beams. Since not enough detail is known about the specimens tested by Thut, Madsen, Peterson, and Moody, recalculation of the regression line coefficients was considered to be unjustified for this present study.

As a result of this foregoing analysis, the determination of an exclusion limit for a working-stress basis was abandoned, since such an approach is irrational in light of the strength-volume effect observed.

## APPLICATION

Current allowable unit stresses for Douglas-fir in tension perpendicular-to-grain are controversial. Canadian and U.S. design practice differs in this respect. Hanrahan (1966) explained why an "interim precautionary measure" should be followed, utilizing a conservative working stress of 15 psi (normal duration of load, dry service) for all loads other than earthquake or wind. This recommendation is still published in the Uniform Building Code (I.C.B.O. 1973), even though the latter now includes the maximum radial-stress formula recommended by Foschi and Fox (1970) and adopted the same year by Canadian code writers (C.S.A. 1970).

In Canada, a working stress of 65 psi is still recommended, pending data that would justify a reduction on a rational basis. However, an interim precautionary measure to reduce the working stress by 50% was subscribed to by many Canadian manufacturers of pitched-tapered beams for most of the 1960s until the maximum radial stress formula was adopted (C.S.A. 1970).

One reason that Canadian code writers have not reduced the allowable unit stress from 65 psi is that Canadian building experience has been relatively good. A recent survey revealed that of more than 1220 pitched-tapered beams erected in Canada between 1955 and 1973, only six have failed in fiber separation. Furthermore, the majority of these 1220 beams were designed by methods less conservative than that required by the maximum radial-stress formula (C.S.A. 1970). Madsen (1972) reasoned that the relatively good Canadian experience might be due to higher live-load-to-dead-load ratios in Canada as compared to those used in the U.S. He suggests that "the loss in strength under continuous loading (is) more critical."

There was either no snow load present at the time of failure of the six beams or it was much less than design load. This phenomenon has yet to be explained. Presence of ring shake in some laminations might be responsible. For the present study, when additional longer specimens were tested, two failed at 12 and 21 psi. These were omitted from the subsequent analysis, since their failures were caused by the presence of ring shake, which most likely existed in the trees before they were felled. They were considered to be atypical because the ring shake was similar in effect to a check not aligned with the applied force direction. Such checks were not admissible because of anticipated difficulty in an evaluation of the size of a check and its effect on strength. The two specimens that failed by ring shake were rejected for the same reason. Inclusion of them would reduce the mean strength of the 5- × 5- × 22-inch group from 126 to 123 psi, increase the standard deviation from 37.4 to 41.3 psi, and affect the regression-line equation slightly.

Since tests by Fox (1974) have produced catastrophic failures of beams, while none of the in-service failures have been of that nature, one suspects that support constraints have been influential. Laboratory tests quoted have been done with linear roller bearings, so that no horizontal thrust can occur. Any thrust resistance provided by beam supports reduces possible maximum radial stresses generated by live load and the probability of catastrophic failures.

Another reason for the difference of opinion between code writers is that no data on tension-perpendicular-to-*glueline* strength tests of Douglas-fir glued-laminated wood were available prior to those of Thut (1970). Although this average and the averages of Table 1 are clearly below those reported for A.S.T.M.-D143 (1965) specimens of clear wood (Kennedy 1965; U.S.D.A. 1955), they are also lower than beam strengths derived by experiment. Fox (1974) reports tests of 12 pitched-tapered beams, most of which failed in bending. Two failed in radial tension at 177 and 286 psi; two others, suspected of radial tension failure, developed 176 and 226 psi. The remaining eight had developed maximum radial stresses ranging from 149 to 230 psi (mean of 191 psi) when they failed in an extreme fiber, thus providing a less-than-maximum potential radial strength. Foschi (1971) reported a single test that yielded an upper bound estimate of 322 psi in maximum radial tension.

The stress distribution within a block badly approximates the conditions in the apex cross section of a pitched-tapered beam carrying a symmetric load. In this cross section, shear stresses are negligible, but there exists a nonlinear distribution of radial tension and tangential stresses (Fox 1970). The presence of parallel-to-grain stresses and the influence of combined stresses are discounted in an assumption that block tests represent beam tests.

The shape and size of specimens are important to results derived. A relationship between relatively inexpensive simple test specimens, such as blocks, and structure-sized pitched-tapered beams is required. This concept has been introduced by Barrett (1974), who has applied the theory of Weibull (1939) to these and other test results. Since tests of blocks have shown that lower strengths are associated with larger volumes, it might be that a size-effect formula should be developed for working stresses in tension perpendicular-to-grain for pitched-tapered beams, as has been done for bending stresses in ordinary beams (U.S.D.A. 1955; Bohannan 1966).

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