DESIGN OF GLUED LAMINATED TIMBER COLUMNS

By John J. Zahn[1] and Douglas R. Rammer[2]

ABSTRACT: The column design criterion adopted in the 1991 National Design Specification (NDS) contains a parameter c that depends on homogeneity and straightness. The objective of this study was to determine whether the improved homogeneity and initial straightness of laminated members are sufficient to justify a more liberal c-value for glued laminated (glulam) compared with that of solid sawn lumber. For Douglas-fir and southern pine glulam columns of four laminations, laminating did not exert a significant effect on c. For Douglas fir, the most likely value of c was found to be 0.76 (probable range, 0.63-0.89). For southern pine, the most likely value of c was 0.83 (probable range, 0.78-0.87). These values do not differ significantly from the value of c = 0.8 adopted for dimension lumber. Therefore, we conclude that laminating does not exert a significant effect on c in glulam of four laminations. However, the NDS column design criterion is still conservative in the design value for compressive strength parallel to grain.

INTRODUCTION

In 1991, the National Design Specification (NDS) for Wood Construction ***(National*** 1991) adopted the Ylinen (1956) column design formula, replacing the 4th power parabola that had been in use, which was based on tests of large timber columns (Newlin and Gahagan 1930). Modern tests of sawn lumber showed the 4th power parabola to be unconservative (Zahn 1986). The new column formula in the 1991 NDS contains a parameter c. If this parameter were adjusted to fit the Newlin and Gahagan (1930) timber column data, it would be 0.97; if the parameter is fitted to modem lumber column data, c would be 0.8. The difference in performance has been attributed to differences in material homogeneity and straightness of members (Zahn 1991).

The objective of this study was to determine the extent to which the performance of lumber under column loading is improved by the effects of laminating the lumber into members of at least four laminations. Although more laminations may show a greater effect, four-lamination members are common in large timber trusses. The measures of performance were crushing strength parallel to grain, buckling strength, and the extent of interaction between these two failure modes. The crushing strength was obtained from tests of very short columns; buckling strength was inferred from the measured modulus of elasticity; and the extent of interaction was measured by the fitted parameter c, which is affected by homogeneity, straightness, and plasticity.

The Ylinen (1956) column formula is a failure model that contains three parameters: zero-length column strength, F_0; buckling strength, F_E; and an interaction parameter, c. If c = 1.0, there is no interaction and the formula reduces to pure crushing and pure buckling. For all real materials, c < 1. The formula has been adopted for use in design by assigning design values. F_c and F_{cE}, in place of F_0 and F_E, respectively. The value of c, however, is not reduced by any safety factor. It is the same in both the failure model and the design model, and it can only be measured by fitting the failure model to mean failure data.

[1]Retired; formerly, Res. General Engr., **USDA** Forest Service, Forest Products Lab.. Madison, WI 53705.

[2]Res. General Engr., USDA Forest Service, Forest Products Lab., Madison. WI.

Note. Associate Editor: Dan L. Wheat. Discussion open until May 1, 1996. To extend the closing date one month. is written request must be filed with the ASCE Manager of Journals. The manuscript (or this paper was submitted for review and possible publication on November 28, 1994. This paper is part of the ***Journal of Structural Engineering,*** Vol. 121, No. 12, December, 1995. ©ASCE, ISSN 0733-9445/95/0012-1789-1794/$2.00 + $.25 per page. Paper No. 9679.

NEW COLUMN DESIGN CRITERION

In the current NDS *(National* 1991), a prime is used to denote that the tabulated design value F_c, has been multiplied by all applicable modification factors, such as load duration, moisture content, and temperature. The resulting quantity is called the "allowable design value ." The effect of slenderness on the allowable design value is accounted for by one modification factor, the column stability factor, C_P The slenderness ratio l_e/d is limited to a maximum value of 50, in which l_e ***is*** effective length and d is the corresponding depth of cross section in the direction of buckling.

The column stability factor C_P is calculated from Ylinen's (1956) formula

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left(\frac{1 + (F_{cE}/F_c^*)}{2c}\right)^2 - \frac{F_{cE}/F_c^*}{c}} \quad (1)$$

in which F_c^* = tabulated compression design value multiplied by all applicable modification factors except $C_F < 1.0$ and C_P (C_F is the compressive size factor for sawn lumber); $F_{cE} = [K_{cE}E'/(l_e/d)^2]$; K_{cE} = 0.3 for visually graded lumber; and K_{cE} = 0.418 for products with $COV_E \leq 0.11$ (where COV is coefficient of variation).

The 1986 NDS column design formula and the new column design formula with several c-values are compared in Fig. 1. The value c = 0.97 nearly matches the 4th power parabola of the 1986 NDS, whereas the value c = 0.8, which was adopted for solid sawn lumber, is considerably more conservative.

Fig. 2 shows the sensitivity of Ylinen's (1956) formula to

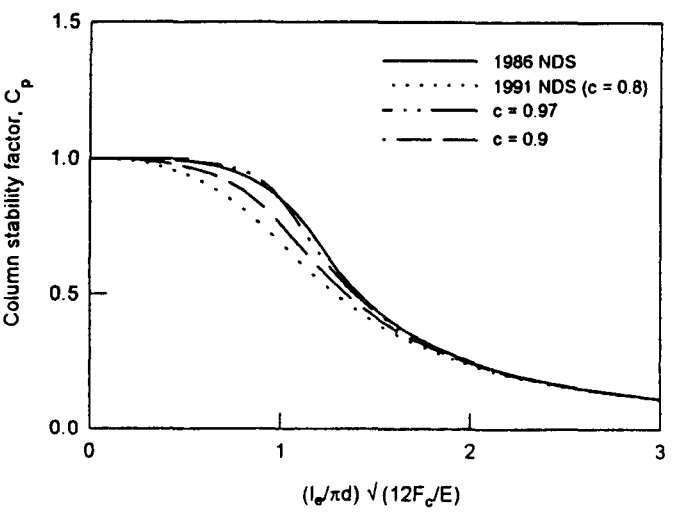

FIG. 1. Comparison of 1991 and 1986 NDS Column Formulas

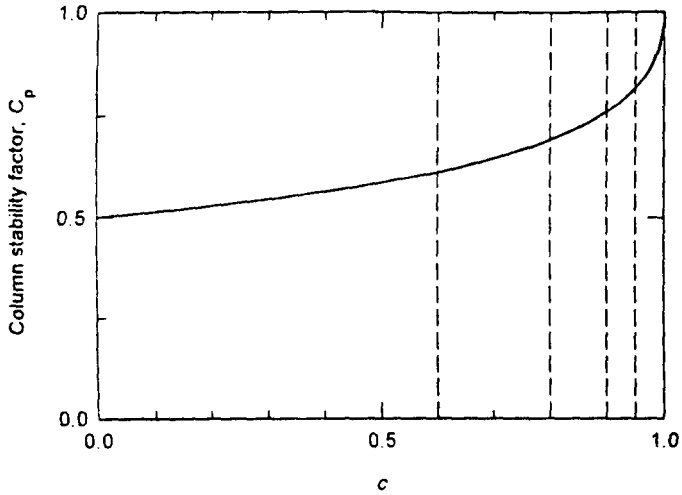

FIG. 2. Sensitivity of Ylinen's Column Formula to Value of Parameter c; Plotted at Point of Maximum Sensitivity

TABLE 1. Experimental Design

Species (1)	Width mm (2)	Width in. (3)	Depth mm (4)	Depth in. (5)	Length m (6)	Length ft (7)	Length/depth (8)	Sample size (9)
Douglas fir	110	4.25	150	6.00	0.30	1	0[a]	88
					1.37	4.5	10	48
					1.68	5.5	12	48
					2.13	7	14	48
					2.44	8	16	48
Southern pine	75	3.00	140	5.50	0.30	1	0[a]	17
					1.83	6	12	9
	130	5.00			0.30	1	0[a]	15
					1.83	6	12	5
					2.13	7	14	8
					2.74	9	18	13
					3.35	(11)	22	17
					3.96	(13)	26	5

[a]These specimens were used to measure zero-length column strength, F_0.

the value of c, plotted for that slenderness at which the formula is most sensitive, namely the slenderness at which the Euler stress equals the compressive strength. The effect is nonlinear. Increasing *c* from 0.8 to 0.9 increases the allowable column load by 10%, whereas decreasing *c* from 0.8 to 0.7 results in a decrease of only 7%.

MATERIALS

All material was derived from the unfailed ends of previously tested large glued laminated (glulam) beams. Two species were available: Douglas fir and southern pine. The Douglas-fir beams were larger and yielded most of the material for this study. Irregular sample sizes were obtained from the southern pine beams. Sample sizes are shown in Table 1.

All the column specimens came from the middle third of the cross sections of the beams. The Douglas-fir laminations were grade L3 *(Standard* 1995) and the southern pine laminations were grade No. 2 *(Standard* 1994). All members were carefully inspected for evidence of cracks, and all suspicious material was rejected. None of the column failures showed evidence of association with prior material failure. Thus, we believe that these laminations, from the center cross sections of outer beam ends. were unaffected by prior beam tests.

The Douglas-fir columns were 150 mm (6 in.) deep and 110 mm (4.25 in.) wide. The southern pine columns were 140 mm (5.5 in.) deep and either 75 mm (3 in.) or 130 mm (5 in.) wide. Individual laminations were 38 mm (1.5 in.) thick (Douglas fir) or 35 mm (1.375 in.) thick (southern pine).

EXPERIMENTAL METHODS

Preliminary Tests

The Ylinen (1956) formula [(1)] reduces to F_c (compressive strength) at zero length. In this paper, we call the zero-length column strength F_0. The 0.30 m (1 ft) members listed in Table 1 were tested in compression, parallel to grain, to obtain the zero-length column strength of the material. Rigid platens were used as end supports, and the head speed was 1.0 mm/s (0.0392 in./s). The distinction between compressive strength and zero-length column strength is further discussed in the section titled "Results."

The flexural modulus of elasticity, *E,* of each column was obtained from a static bending test with loads applied at the third points. Bending stresses ***were*** kept less than 3.5 MPa (500 lb/in.^{2}). Load was applied in the strong axis direction (perpendicular to the wide face of laminations), with the other direction supported to prevent lateral buckling. The head speed was sufficient to reach the desired maximum stress in approximately 5 min. Note that ***E*** varied systematically with the length of the member (Fig. 3), indicating a possible effect of shear deformation on ***E.*** Had we used an overall average ***E*** for each species, the inferred c-values would have been lower than those reported here, (In Fig. 3, there is an increasing error with a shorter length.)

Column Tests

All members were laterally supported at the third points to prevent buckling in the weak direction. This simulated the lateral support given to truss chords by the attached decking. End supports were on rollers equivalent to a pinned-end condition (Fig. 4). The head speed was equal to the length of the specimen divided by 1,000 s.

Column tests were delayed until the results of all preliminary tests were available. Knowledge of ***E*** and F_c^* allowed the column length to be selected so that the Euler stress was approximately equal to the crushing strength. This ensured that the column tests would fall in the range where Ylinen's (1956) formula is most sensitive to *c.* The column lengths shown in Table 1 were deemed suitable to determine *c.*

Exploratory testing of scrap material revealed that dithering was necessary. Dithers are vibrators that supply the energy needed to break static friction. Under a heavy axial

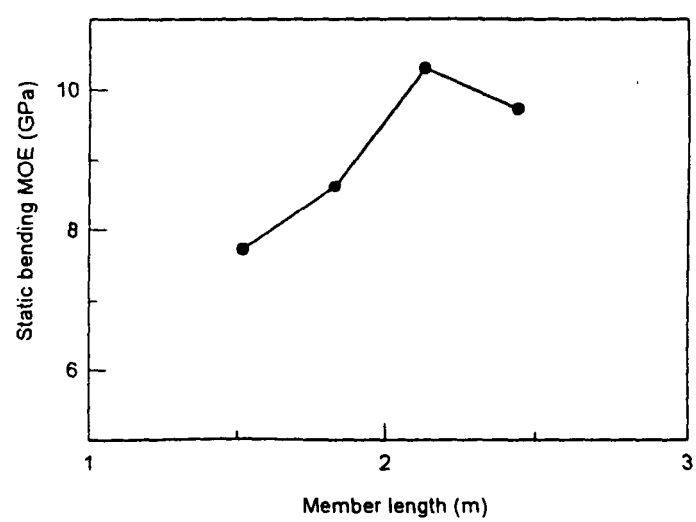

FIG. 3. Average Measured Modulus of Elasticity as Function of Member Length for Douglas Fir

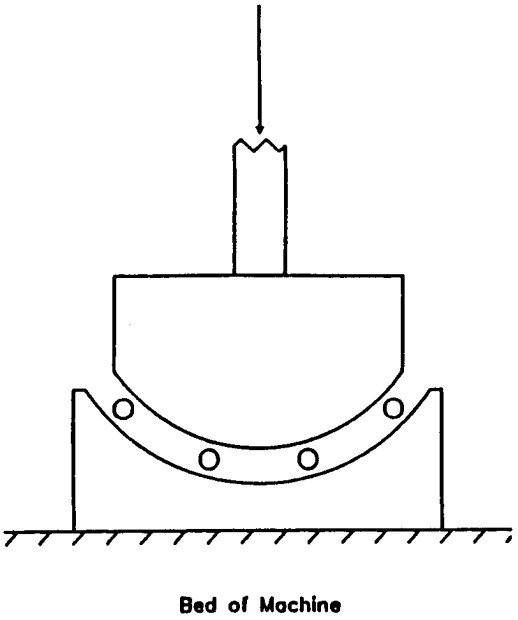

FIG. 4. End-Roller Support Used to Obtain Condition of Simple Support (Pinned Ends)

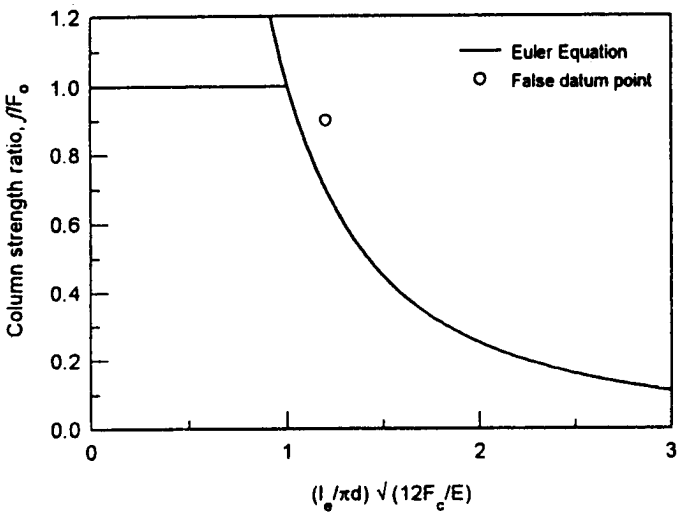

FIG. 5. Typical False Datum Obtained When Rollers in End Supports Locked up from Static Friction under Heavy Axial Load

load, the rollers in the end supports (Fig. 4) would sometimes lock up. If the member were centered very accurately and its cross sections had good material symmetry, enough rolling friction could develop to make the test behave like a test of square ends on rigid platens rather than a test of simple support. Friction between the specimen and lateral supports could also have been a factor to prevent buckling. During these exploratory tests without dithering, the column load would often exceed the Euler load (Fig. 5) and, in rare cases, it reached nearly as high as F_o.

For simply supported columns, the Euler load is always an upper bound on the real column capacity. Further, in real structures the end condition is more like that of simple support than that of a rigid platen. Therefore, a vibrator was attached to the bottom support to prevent the rollers from locking up. This gentle vibration was also sufficient to break static friction at points of lateral support. Dithering eliminated all occurrences of loads in excess of the Euler load.

RESULTS

Compressive Strength Tests

Mean zero-length column strength values, (F_{0P}) were inferred from AITC 117-93 (*Standard* 1993), the published design values, as follows. According to the ASTM D3737 (*Standard* 1994a) standard. multiplying the published design stress by 1.9 should give the 5th percentile strength. Assuming a normal distribution, the mean can be inferred from the 5th percentile by the following relation:

$$\text{mean} = \frac{\text{5th percentile}}{1 - 1.645\ \text{COV}} \tag{2}$$

Data from the in-grade program (Green and Evans 1987) suggest that the COV of compressive strength is approximately 0.20 for No. 2 Douglas fir and 0.16 for No. 2 southern pine. The column specimens as tested had laminations that were all of one grade. The results were compared to axial combinations listed in ***AITC 117-93*** (*Standard* 1993): No. 1 and No. 47 axial combinations for Douglas fir and southern pine, respectively. Values for design stress in compression, parallel to grain, for these combinations are 10.7 MPa (1.55 $\times 10^3$ lb/in.^{2}) for Douglas fir and 13.1 MPa (1.90 $\times 10^3$ lb/in.^{2}) for southern pine. Therefore, (2) gives the estimated mean compressive strength of L3 Douglas fir as

$$F_{0p} = \frac{10.7(1.9)}{1 - 1.645(0.2)} = 30.3\ \text{MPa}\ (4{,}390\ \text{lb/in.}^2) \tag{3}$$

and that of No. 2 southern pine as

$$F_{0p} = \frac{13.1(1.9)}{1 - 1.645(0.16)} = 40.2\ \text{MPa}\ (5{,}840\ \text{lb/in.}^2) \tag{4}$$

The mean values of zero-length column strength, F_0, are shown in Table 2 along with their COV values and the estimated mean compressive strength, F_{0p}. Table 2 also shows low and high values of measured strength, F_0, obtained from the mean plus or minus one standard deviation. In all cases, experimental results exceeded the adjusted estimated mean compressive strength (F_{0p}) values. This phenomenon was a direct result of test methods used to determine F_0 and F_{0p}.

Current published parallel-to-grain compressive strength (F_c) values ignore both the length effect and laminating effect. Consider first the length effect. The F_c value is based on lumber strength values derived from either tests of short members with the worst defect (based on grading rules) or tests of long members laterally supported at 0.30 m (1 ft) intervals (*Standard* 1994b). In either case, the strength is governed by the worst defect. However, random shorter members have a smaller likelihood of containing a defect and are inherently stronger. Length effects of this type have been previously observed loaded in tension. Studies by Showalter et al. (1987) showed that shortening a test specimen from 3.05 m (120 in.)

TABLE 2. Results of Short-Block Compressive Strength Tests

Species (1)	Test numbers (2)	F_0 (MPa) Low[a] (3)	Mean (4)	High[b] (5)	COV (6)	F_{0p} MPa (7)	lb/sq in. × 10³ (8)
Douglas fir	88	37.4 (5.43)[c]	43.5 (6.31)[c]	49.5 (7.18)[c]	0.139	30.3	4.39
Southern pine	35	45.2 (6.56)[c]	50.4 (7.32)[c]	55.7 (8.08)[c]	0.104	40.2	5.84

[a]Mean minus one standard deviation.
[b]Mean plus one standard deviation.
[c]lb/sq in. × 10³.

TABLE 3. Effects of Laminating on Mean, 5th Percentile, and Design Values[a]

Grade and species (1)	Percent Change in Compression Strength from Laminating: Mean[b] (2)	5th percentile[c] (3)	AITC Design Values[d]: MPA (4)	lb/sq in. × 10^3 (5)	Ratio to design[e] (6)
L3 Douglas fir	+24 (±9)	+61	8.27	1.20	1.47
No. 2 southern pine	+13 (±8)	+28	7.92	1.15	1.65
No. 2 southern pine (1.6E)	+11 (±5)	+21	7.92	1.15	1.83

[a]Moody (1981).
[b]Values in parentheses are for 95% confidence interval.
[c]Calculated assuming normal distribution with 75% confidence at 5th percentile.
[d]Values given in AITC-93 assuming medium grain.
[e]5th percentile values/1.9 divided by AITC design values.

TABLE 4. Results of Tests on Modulus of Elasticity

Species (1)	Measured Mean: GPa (2)	lb/sq in. × 10^6 (3)	COV (4)	Design Mean[a]: MPa (5)	lb/sq in. (6)
Douglas fir, L3	9.09	1.32	0.175	10.3	1.5
Southern pine, No. 2	10.0	1.52	0.133	9.65	1.4

[a]Standard (1993).

to 0.62 m (30 in.) increased tension, parallel to grain, by at least 16%.

The laminating effect also increases strength. Where defects do occur. they are locally reinforced by defect-free adjacent laminations. Moody (1981) investigated laminating effects on long compression specimens constructed with various grades of Douglas-fir and southern pine laminating material. In comparing the parallel-to-grain compression strength of single members and two-ply laminated members, Moody (1981) concluded that laminating significantly increases compressive strength. Not only was the mean strength increased, but the coefficient of variation was reduced, resulting in a substantial increase in design values. Table 3 lists the percent change in mean strength and 5th percentile values, American Institute of Timber Construction (AITC) design values, and design ratios, which indicates how much design values would be increased by taking the laminating effect into account. In all cases, the design values could be increased at least 50%.

In this study, the zero-length column strength F_0, was obtained from tests of a 0.30 m (1 ft) laminated column without visibie defects. Therefore, F_0 values were significantly greater than the estimated mean compressive strength values (F_{0p}) obtained from published allowable values because of length and laminating effects

Modulus of Elasticity Tests

Measured modulus of elasticity values of the tested columns are shown in Table 4. Published values (design mean values) are shown for comparison. The published modulus of elasticity values of glued, laminated (glulam) members with four or more laminations is 10.3 GPa (1.5 x 10^6 lb/in.^{2}) for L3 Douglas fir and 9.0 GPa (1.3 × 10^6 lb/in.^{2}) for No. 2 southern pine (*Standard* 1993).

Column Tests

For each column, width b. depth d, length l, and failure load P were recorded. Column strength f and Euler stress F_E, were calculated as follows:

$$f = \frac{P}{bd}; \quad F_E = \frac{\pi^2 E d^2}{12 l^2} \qquad (5, 6)$$

All data for a given species were plotted on a single figure of f/F_0 versus $\sqrt{F_0/F_E}$ [that is, $(l/\pi d)\sqrt{12F_0/E}$]. The scatter on such a figure shows the variability in f, but it does not reflect the variability in F_0 and F_E (Figs. 6 and 7). These figures show the best-fitting Ylinen (1956) formula for comparison, with c obtained by nonlinear least squares. Most likely c- values for Douglas fir and southern pine are 0.76 and 0.83, respectively. Standard error of the c- values was 0.015 for Douglas fir and 0.024 for southern pine.

To show the influence of F_0 on c, a least-squares fit was conducted assuming F_0 values of plus or minus one standard deviation from the mean (Fig. 8). In Fig, 8. solid circles represent mean values and open circles represent a one standard deviation shift of F_0. A smaller value of F_0 increases the fitted vaiue of c and vice versa. For Douglas fir, probable c-values lie within a range of 0.63-0.89; for southern pine, probable c-values lie within a range of 0.78-0.87. The influence of modulus of elasticity on c is shown in a similar way in Fig.

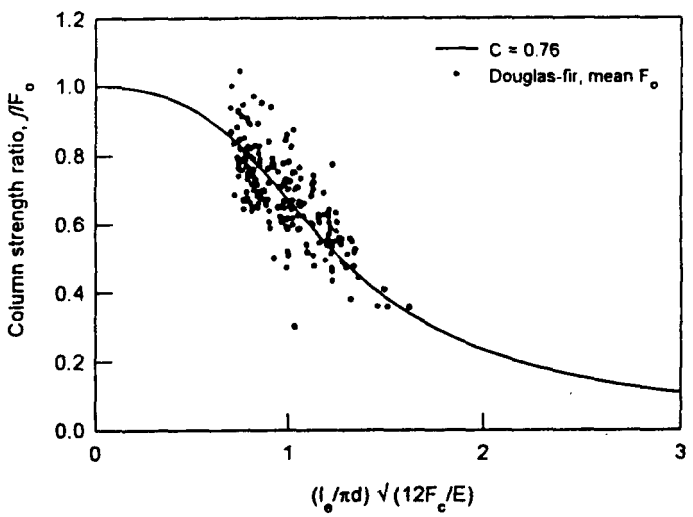

FIG. 6. Results of 192 Douglas-Fir Column Tests, Based on Assumption that F_c = 37.4 MPa (Mean Value)

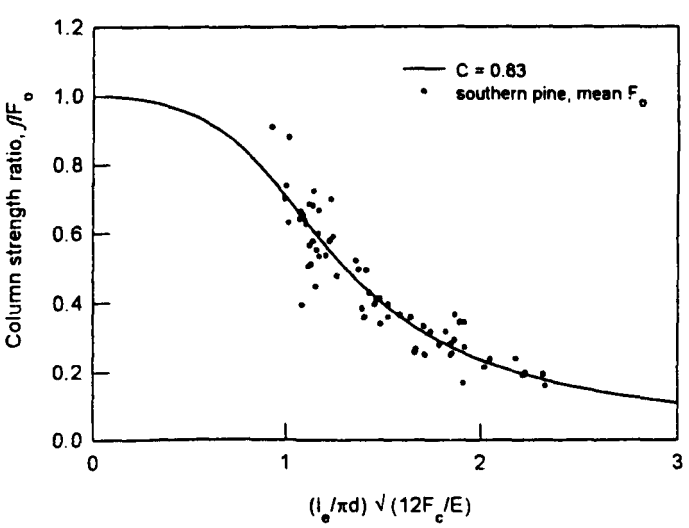

FIG. 7. Results of 89 Southern Pine Column Tests, Based on Assumption that F_c = 50.4 MPa (Mean Value)

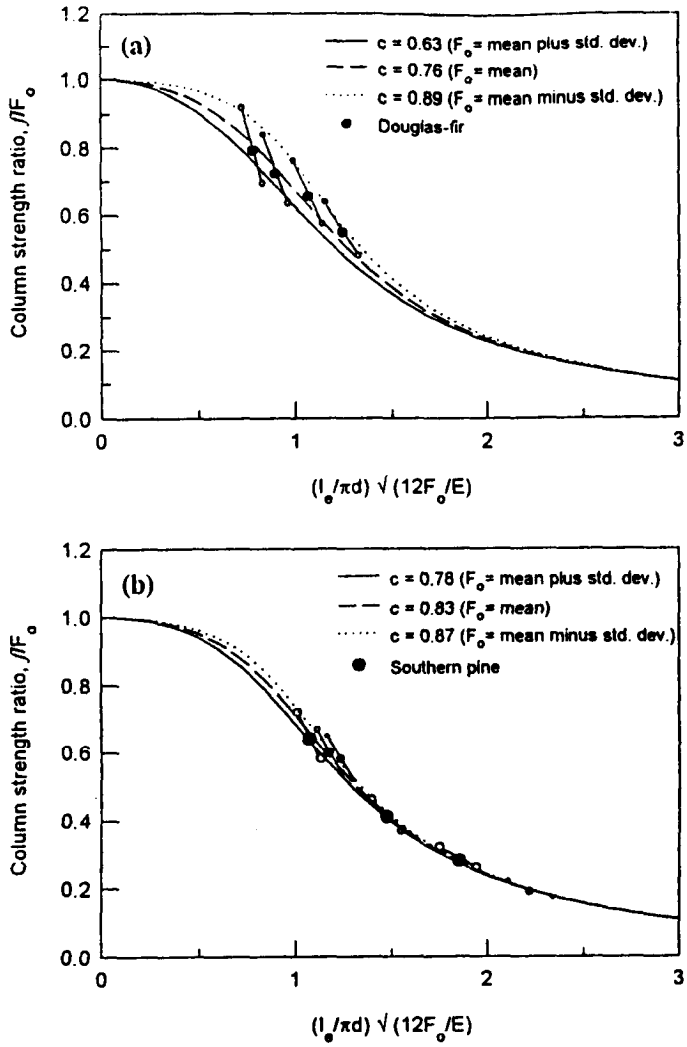

FIG. 8. Effect of Variability in F_0 on Fitted c-Value: (a) Douglas-Fir Columns; (b) Southern Pine Columns

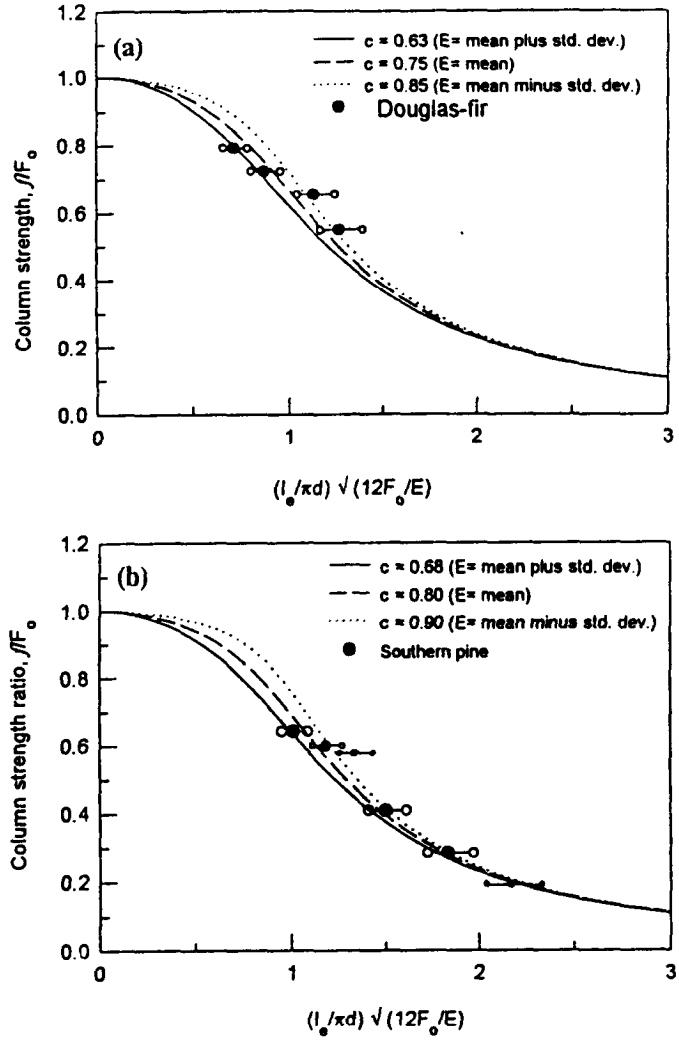

FIG. 9. Effect of Variability in *E* on Fitted c-Value: (a) Douglas-Fir Columns; (b) Southern Pine Columns

9. Again, *c* was fitted by least squares and ***E*** was varied by plus or minus one standard deviation. The variabilities in F_0 and *E* have approximately the same effect on *c*. (In both Figs. 8 and 9, the marker size is congruent with the sample size.)

Implications for Column Design

The ***c*** values obtained here for glulam do not differ significantly from the value of $c = 0.8$ adopted for dimension lumber in the current NDS *(National* 1991). Therefore, we conclude that laminating does not exert a significant overall effect on *c* in the glulam of four laminations. This result is most evident in Fig. 10, in which sample averages of both species are plotted with Ylinen's (1956) formula at $c = 0.8$. In this figure. the relative size of the southern-pine samples is indicated by a relative marker size. (In Fig. 10, the solid line is the current failure model for lumber.)

This result was not anticipated. The value of $c = 0.9$ in the current NDS *(National 1991)* was chosen in the belief that glulam would have a higher c-value than dimension lumber because of its greater homogeneity and improved straightness. We now believe that the laminating effect on *c* is the product of two effects: improved homogeneity, which increases *c*. and the laminating effect on strength. which decreases *c*. The laminating effect on strength is caused by increased plasticity; that is, by the fact that the increased failing stress exceeds the proportional limit. The two effects are essentially equal and opposite, resulting in a *c*- value for glulam

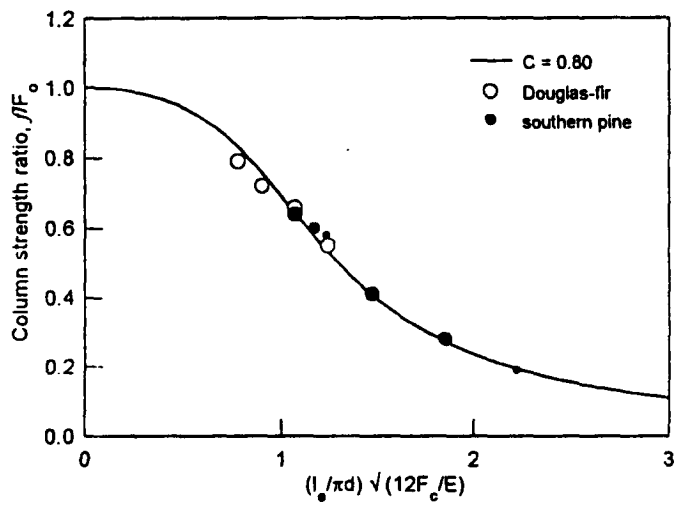

FIG. 10. Summary of Results, Marker Size for Southern Pine is Congruent with Sample Size

that is approximately equal to the c-value for the dimension lumber from which it is made.

The current NDS *(National* 1991) has not been unsafe. It is conservative at all column lengths, in spite of having a too-liberal value of $c = 0.9$, because the design value of F_c ***is*** conservatively low as a result of not taking into consideration laminating and length effects on compressive strength.

CONCLUSIONS AND RECOMMENDATIONS

For glulam columns with only four laminations. laminating exerts only a small effect on c. This study showed that the most probable value of c for glulam with four laminations was 0.76 for Douglas fir and 0.83 for southern pine, with the larger F_c values. This result is in general agreement with the value $c = 0.8$ adopted for lumber. Additional laminations may show more of an effect, but it would be awkward to use a different failure model (i.e., assign a different c- value) for different numbers of laminations. Further, four laminations is a common construction.

Laminating may exert an effect on the compressive strength, F_c, as suggested by the large difference between measured F_0 and the design mean inferred from the published F_c.

These data show a small difference in c between Douglas-fir and southern pine glulam, but it does not appear to be significant. Small differences in c are difficult to measure experimentally. and it would be awkward to assign c according to species in design.

APPENDIX I. REFERENCES

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APPENDIX II. NOTATION

The following symbols are used in this paper:

b = width of cross section;
C_F, C_P = modification factors in the 1991 NDS;
c = Ylinen parameter;
d = depth of cross section;
E = modulus of elasticity;
F_c = compressive strength;
F_{cE} = allowable buckling stress in the 1991 NDS;
F_E = Euler stress;
F_0 = zero-length column strength;
f = column strength;
K_{cE} = reduction factor on E in the 1991 NDS;
l, l_e = length of simply supported column, or equivalent length; and
P = column capacity.

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