STRUCTURAL CHARACTERIZATION OF BUILT-UP TIMBER COLUMNS

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ABSTRACT: The use of novel built-up columns as structural members is investigated. The columns are fabricated from dimensional timber and connected with mechanically applied adhesive, using a proprietary system. An extensive test program was carried out on full-scale built-up timber columns. Axial, flexural, and squash load tests, intended to characterize the service and ultimate behaviors of the columns, are reported. The results of the test program are compared with standard calculations of allowable design capacities for non-built-up timber columns. It is concluded that the built-up columns tested may be safely designed using the same methods as are used for solid timber columns that use the material properties of the constituent timbers. This results in a significant advantage in load-carrying capacity of the built-up sections over solid sections for columns having dimensions greater than 127 × 127 mm (5 × 5 in.).

INTRODUCTION

In many applications, timber columns are built up from smaller dimensional lumber and connected with fasteners or adhesive. Such built-up columns may be solid or have a hollow square cross section. Traditional box columns are a specific type of built-up column in which four component members are placed around the periphery of the section, leaving a hollow core in the middle. Fig. 1(a) shows the typical arrangement of a timber box column. These columns are normally assembled with mechanical fasteners since their form does not lend itself easily to the use of adhesives.

An alternative arrangement of timbers in a built-up column is proposed. This arrangement of dimensional lumber, shown in Fig. 1(b), easily lends itself to automated fabrication using adhesives. This arrangement, however, is not biaxially symmetric as is the typical arrangement. It is hypothesized, therefore, that the built-up columns may have a strong and a weak axis. Furthermore, the lack of mechanical connectors requires that the adhesive transfer all shear forces in the section.

Hollow sections derive their efficiency from placing the material around the periphery of the section where it is most efficient in resisting buckling loads. A hollow section has a greater radius of gyration than a solid section having the same cross-sectional area and is thus inherently more stable for a given cross-sectional area.

Unfortunately, very little guidance is available for the design of box columns. Available standards consider built-up (or composite) members only inasmuch as they provide sufficient connection between the component timbers. Essentially, the design of built-up sections amounts to selecting an appropriate spacing for the connection hardware used. Built-up sections fabricated using adhesives are often left to the designer to determine appropriate methods of treating the design of the column. In this regard, an informal review of adhesive manufacturers arrived at the consensus that "the glue is always stronger than the timber."

Clause 3.6.2.3 of the *1997 National Design Specification for Wood Construction* (AF&PA 1997) states, in part, the following: "Individual laminations of mechanically laminated built-up columns shall be designed in accordance with [clauses relating to solid columns] . . ." It is felt that this guidance may be unduly harsh if applied to the built-up columns being considered.

Additionally, there is very little literature available pertaining to timber box columns. The available literature mostly discusses mechanically connected box columns and the role that the mechanical connections play in the behavior of the column. Van Dyer (1992) presents a model for determining the allowable axial stress in a box column of varying length. Van Dyer's model is directed toward optimization of mechanical connectors (nails, in this case) and considers the column geometry shown in Fig. 1(a). As an indication of the efficiency of a box column, Van Dyer reports that a 3 m (10 ft) column having excellent connection details has an axial stress carrying capacity approximately 1.8 times that of a solid column having the same sectional area.

Other researchers (Neubauer 1972 and Kinzey 1951, among others) have also addressed mechanically connected box columns. There appears to be no literature available pertaining to adhesively connected box columns.

The advantages of the built-up columns considered in this study are as follows:

1. Efficient use of material, allowing significantly greater loads to be carried and higher allowable stresses to be used
2. Improved quality control stemming from the use of smaller dimensional lumber sections
3. Improved quality control through the use of an automated fabrication procedure not suitable for traditional box columns
4. Reduced cost-to-column capacity ratio

The objective of the present study is to characterize the behavior of the built-up columns and to provide guidance to

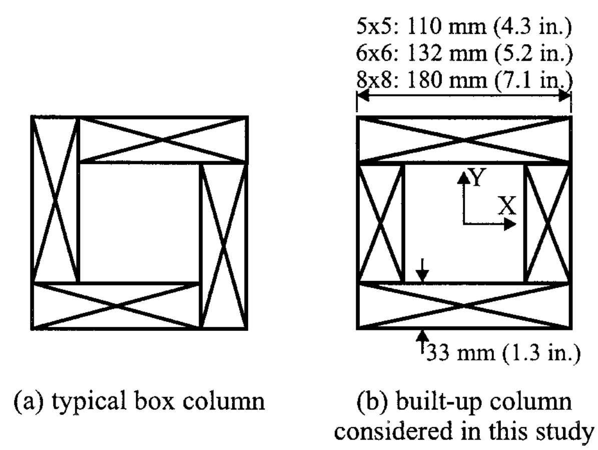

FIG. 1. Built-Up Box Columns and Nominal Dimensions

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designers wishing to use these built-up columns. In order to provide useful comparisons for the practitioner, experimental column behavior is measured alongside that predicted by the NDS.

DESIGN LOADS OF BUILT-UP COLUMNS

Allowable Axial Design Loads

The following summarizes the determination of allowable axial load that may be applied to pressure-treated Southern Pine composite columns. The determination of allowable axial stress is based on the *1997 National Design Specification for Wood Construction* (NDS).

The composite columns considered are oriented such that the compression forces are parallel to the grain. The allowable compression stress parallel to grain, f_c, is given by NDS Clause 3.6.3 as

$$f_c \leq F_c C_D C_M C_t C_F C_i \tag{1}$$

where F_c = tabulated allowable compressive stress parallel to the grain, and C_D, C_M, C_t, C_F, and C_i are factors reflecting load duration, wet service conditions, temperature, member size, and incision, respectively. For the foregoing discussion, the above factors are all considered to be equal to 1.0. In addition to the adjustment factors in (1), a column stability factor, C_P, is used to account for slenderness effects of long, unbraced rectangular solid columns. The value of C_p is given by NDS Clause 3.7.1 as

$$C_P = \frac{1 + (F_{cE}/f_c)}{2c} - \sqrt{\left(\frac{1 + (F_{cE}/f_c)}{2c}\right)^2 - \frac{(F_{cE}/f_c)}{c}} \tag{2}$$

where

$$F_{cE} = \frac{0.3E}{(L_e/d)^2} \tag{3}$$

where E = tabulated value of Young's modulus; L_e = effective length of the column, KL; d = smaller dimension of a rectangular column section; and c is taken as 0.80 for sawn timber.

Thus, for rectangular columns with compression parallel to the grain and a slenderness ratio, L_e/d, less than 50, the allowable design compressive stress, F_c', is found to be

$$F_c' = F_c\left[\frac{1 + (F_{cE}/f_c)}{2c} - \sqrt{\left(\frac{1 + (F_{cE}/f_c)}{2c}\right)^2 - \frac{(F_{cE}/f_c)}{c}}\right] \tag{4}$$

Appendix H: *Lateral Stability of Columns* of the NDS document summarizes the basis for determining the allowable compressive stress outlined above as follows. Eq. (3) is derived from the Euler buckling formulation for solid rectangular members converting the radius of gyration, r, to the weak axis depth of the section, d. Hollow rectangular sections, however, are more efficient since they have a proportionally larger radius of gyration for the same sectional area. Additionally, the 0.3 factor includes a reduction factor of 2.74 from the tabulated modulus of elasticity design values. Together, these equations represent the approximate 5% lower exclusion value for buckling with a 1.66 factor of safety.

The NDS equations are based on solid rectangular sawn timber columns. The columns considered in this study are hollow built-up square columns formed from sawn timber. It is proposed, nonetheless, that these equations are valid for the present case.

The use of the NDS equations makes the assumption of a "solid" column. This implies that the continuity between the column components is maintained through buckling. The question that arises is, therefore, whether the adhesive provides sufficient bond to permit the individual components of the column to be considered as continuously supported against lateral buckling. In general, the restraint required to provide "continuous support" to a member is relatively small. It is unlikely that the adhesive capacity will be overcome in concentric axial load tests.

EXPERIMENTAL PROGRAM

An experimental program (Harries and Petrou 1999) involving 11 concentric axial load tests of full-length columns, 18 midpoint flexural tests, and 13 squash load tests of column stubs was conducted. The objectives of the program were

1. To determine the maximum allowable design load and thus to verify the applicability of the NDS design equations for use with built-up columns
2. To characterize the ultimate behavior of the built columns through flexural and squash load tests

Specimen designations (4 × 4, 5 × 5, 6 × 6, and 8 × 8) used in this paper refer to the nominal column dimensions in inches corresponding to typical North American lumber dimensions. Equivalent English dimensions provided throughout this paper are given in the same *nominal* dimensions typical of North American timber practice.

Test Specimens

Four sizes of pressure-treated, built-up sawn timber columns were tested. All column sections were fabricated from 33 mm (2 in., nominal) sawn timbers glued together using a Resorcinol adhesive. Table 1 gives the nominal column dimensions. Fig. 1(b) shows the typical column cross section and the manner in which the component timbers were assembled to form the built-up column. The columns designated 4 × 4 are solid built-up columns equivalent in size to a sawn timber 89 × 89 mm (4 × 4 in., nominally). These columns are made up of a pair of 33 × 89 mm (2 × 4 in.) sawn timbers sandwiching a thinner 19 × 89 mm (3/4 × 4 in.) sawn timber.

The predicted axial capacities of the column specimens are given in Tables 1 and 2. These capacities are calculated based on the NDS equations presented in the previous section. The values presented in the top section of Table 1 are for columns having an effective length factor K = 1 [see (3)]. It is noted that for safety reasons, the structural tests reported here were conducted with fixed-pinned end conditions (K = 0.7). Common practice (including NDS Appendix G: *Effective Column Length*) recommends that a value of K = 0.8 be used to account for "imperfect" end conditions. The lower section of Table 1 reports the critical axial loads determined using a value of K = 0.80.

Similarly, NDS prescribed equations were used to determine the predicted allowable flexural and shear capacities for the columns tested; these are summarized in Table 3.

Axial Load Test Set-Up, Instrumentation and Protocol

Columns were subject to a monotonically increasing concentric axial load. Instruments record the applied load, axial shortening of the column, and lateral deflection in both principal directions at the midheight of the column.

The self-reacting axial test setup is shown in Fig. 2(a). Load was applied using a 222 kN (50 kip) hydraulic ram at the "top" of the column and was reacted by a pair of 152 × 102 × 13 (6 × 4 × 1/2) hollow steel strongbacks spanning a pair of 44 mm (1 3/4 in.) diameter high-strength threaded rods. To prevent instability of the test apparatus about the ball joint,

TABLE 1. Dimensions and Allowable Axial Load Calculations for Built-Up Columns

Dimension (1)	Symbol (2)	5 × 5 (3)	6 × 6 (4)	8 × 8 (5)	4 × 4 solid (6)
(a) Using Length Factor, K = 1.0					
Nominal dimension, mm	d	109	132	180	89
Nominal wall thickness, mm	t	33	33	33	n.a.
Column height, mm	L	3,048	3,048	3,048	3,048
Sectional area, mm^2	A	10,064	13,097	19,484	9,935
Moment of inertia, mm^4	I	11.57×10^6	23.77×10^6	48.95×10^6	5.20×10^6
Radius of gyration, mm	r	34	43	62	26
Compression parallel to grain,[a] MPA	F_c	13.79	13.79	13.10	14.48
Modulus of elasticity,[a] MPa	E	12,400	12,400	12,400	12,400
Critical buckling stress, MPa	F_{cE}	4.78	7.00	13.03	3.16
Column stability factor	C_P	0.32	0.44	0.69	0.21
Allowable design stress, MPa	F'_c	4.37	6.05	9.03	3.10
Allowable design axial load, kN	P	44.0	79.1	175.7	23.8
(b) Using Length Factor, K = 0.8					
Critical buckling stress, MPa	F_{cE}	7.47	10.92	13.10[b]	4.95
Column stability factor	C_P	0.46	0.61	0.59	0.31
Allowable design stress, MPa	F'_c	6.37	8.36	9.05	4.54
Allowable design axial load, kN	P	64.1	109.3	176.1	35.8

[a]Values from NDS *Supplement* Table 4B for select structural grade Southern Pine sawn timber.
[b]Buckling stress limited by allowable compressive stress parallel to grain, F_c.

TABLE 2. Axial Load and Deflection Results

	NDS Allowable Load, kN		Initial Buckling		Ultimate Observed Load			Overstrength Ratio	
Specimen (1)	K = 1.0 (2)	K = 0.8 (3)	Load (kN) (4)	Axial deflection (mm) (5)	Load (kN) (6)	Axial deflection (mm) (7)	Elastic buckling (8)	K = 1.0 (9)	K = 0.8 (10)
5 × 5-B	44.0	64.1	73.3	2.67	110.0	4.55	*x-x*	2.50	1.71
5 × 5-C	44.0	64.1	69.1	2.14	103.1	4.19	*y-y*	2.34	1.61
5 × 5-G	44.0	64.1	77.9	1.57	146.6	2.90	*x-x*	3.33	2.29
5 × 5-H	44.0	64.1	87.0	1.93	116.8	2.36	*x-x*	2.65	1.82
6 × 6-E	79.1	109.3	—[a]	—[a]	174.1	2.18	—[a]	>2.20	>1.59
6 × 6-F	79.1	109.3	—[a]	—[a]	183.2	4.37	—[a]	>2.32	>1.68
8 × 8-E	175.7	176.1	—[a]	—[a]	217.6	2.95	—[a]	>1.24	>1.24
8 × 8-F	175.7	176.1	—[a]	—[a]	217.6	2.79	—[a]	>1.24	>1.24
4 × 4-A	23.8	36.0	32.0	1.85	61.8	—	both	2.60	1.73
4 × 4-B	23.8	36.0	34.4	1.55	57.3	3.40	both	2.41	1.60

[a]No buckling observed.

TABLE 3. Flexural Load and Deflection Results

		NDS Allowable Loads					Overstrength Ratio		
Specimen (1)	Flexural axis (2)	Flexure (kNm) (3)	Shear (kN) (4)	Ultimate shear (kN) (5)	Ultimate moment (kNm) (6)	Midspan deflection (mm) (7)	Flexure (8)	Shear (9)	Ultimate limit state (10)
5 × 5-D	*x-x*	3.7 (shear equal to 3.0 kN)	6.2	10.2	12.5	50	3.4	>1.7	flexure (tension flange)
5 × 5-E	*x-x*			12.9	15.7	66	4.2	>2.1	flexure
5 × 5-F	*y-y*			8.7	10.6	43	>2.8	1.4	shear in web timber
5 × 5-J	*x-x*			12.0	14.6	47	3.9	>1.9	flexure (at discontinuity)
5 × 5-K	*x-x*			12.4	15.2	84	4.1	>2.0	flexure
5 × 5-L	*y-y*			12.0	14.6	55	3.9	>1.9	flexure
5 × 5-M	*y-y*			19.1	12.5	51	3.4	>1.7	flexure
6 × 6-A	*x-x*	6.3 (5.2)	7.7	22.8	27.8	64	4.4	>3.0	flexure (tension flange)
6 × 6-B	*x-x*			20.0	24.4	69	3.9	>2.6	flexure (compression flange)
6 × 6-C	*y-y*			21.7	26.4	43	>4.2	2.8	shear of web timber
6 × 6-D	*y-y*			20.0	24.4	44	3.9	2.6	shear at discontinuity
8 × 8-A	*x-x*	13.0 (10.6)	11.1	23.7	29.1	23	2.2	2.1	flexure initiated at knot
8 × 8-B	*x-x*			23.7	29.1	25	>2.2	2.1	shear of web timber
8 × 8-C	*y-y*			31.7	38.6	28	3.0	>2.9	flexure (web timber)
8 × 8-D	*y-y*			23.7	29.1	26	>2.2	2.1	shear of web timber
4 × 4-C	*x-x*	2.3 (1.9)	6.5	3.1	3.8	55	1.6	>0.5	flexure initiated at knot
4 × 4-D	*x-x*			7.0	8.5	65	3.7	>1.1	flexure
4 × 4-E[a]	*x-x*			16.0	9.8	25	4.2	2.5	shear/flexure failure

[a]Specimen 4 × 4-E was tested with a span of 1,220 mm (48 in.).

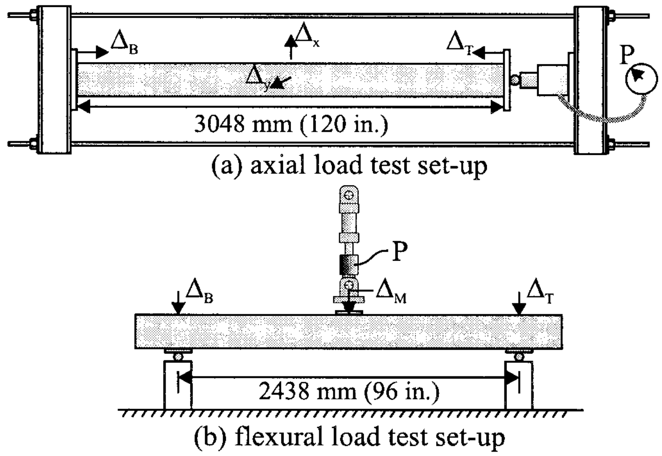

FIG. 2. Axial and Flexural Test Setups and Instrumentation

the body of the hydraulic ram was braced horizontally and vertically exclusive of the self-reacting frame.

Self-centering steel platens were located at both the top and bottom of the column. These were capped into place using plaster-of-Paris to ensure uniform distribution of load over the entire column cross section. It is noted that timber dimensions are not exact and some eccentricity is inevitable. Based on the test setup design and the dimensions of the columns, the maximum likely eccentricity of axial load is 3 mm (1/8 in.) for the 5 × 5 and 6 × 6 columns and 6 mm (1/4 in.) for the 8 × 8 columns. This represents a 3, 2.5, and 3.5% eccentricity, respectively. Additional eccentricities may result from initial out-of-straightness or twist of the column specimens.

Although it would have been optimal to provide pinned-pinned end conditions for the columns, doing so would result in the test setup being inherently unstable. The end conditions provided were fixed-pinned, theoretically resulting in a column effective length factor K equal to 0.70. It is felt that because of the imperfect end conditions provided, a column effective length factor K equal to 0.80 reasonably represents the test conditions. It is noted here that an effective length factor of 0.80 is probably a good representation of square-cut timber columns in simple bearing. The NDS recommends a value of K equal to 1.0 (pinned-pinned) for this case, with the acknowledgment that this is a conservative value.

Axial and lateral deflections at midheight were measured using dial gauges having precision of 0.025 mm (0.001 in.). The gauge locations are shown in Fig. 2. The arithmetic difference between the top and bottom axial deflections was used to determine the axial shortening of the column. The lateral deflection values were a direct measure of out-of-plane buckling of the column. The applied axial load was determined from the hydraulic pressure driving the ram with a resolution of 690 kPa (100 psi).

The specimens were incrementally loaded in load control until buckling becomes apparent. Buckling was evidenced by continuously increasing lateral deflections (Δ_x and/or Δ_y) without a corresponding increase in applied load. The onset of buckling was noted, beyond which deflection readings were no longer possible to accurately obtain. Beyond the onset of buckling, the axial load was monotonically increased at a rate of approximately 445 N (100 pounds) per second. A visual (and aural) inspection of the column was maintained through this process and significant observations were noted.

Flexural Test Set-Up, Instrumentation and Protocol

Columns were subject to a monotonically increasing load at their midspan. The columns were simply supported on 2,438 mm (96 in.) centers. Specimen 4 × 4-E was tested with a span of 1,200 mm (48 in.). Instruments recorded the applied load, midspan deflection, and the deflections at each support.

The test setup is shown in Fig. 2(b). Load was applied using a 222 kN (50 kip) hydraulic actuator located at the midspan of the column. This actuator was positioned in a self-reacting frame posttensioned to the strong floor. Support reactions are applied directly through the strong floor. Rollers were provided at each support and at the midspan loading location. Steel plates were provided between the rollers and the timber columns in order to distribute the concentrated bearing forces and avoid perpendicular crushing of the timbers at these locations. These plates were capped in place with plaster-of-Paris to ensure uniform load distribution.

Deflections at midspan and at each support were measured using dial gauges having precision of 0.025 mm (0.001 in.). The gauge locations are shown in Fig. 2. The arithmetic average of the support deflections was subtracted from the recorded midspan deflection to determine the midspan deflection relative to the ends of the columns (i.e., the real deflection seen by the column). The applied load was recorded from a load cell, having a resolution of 222 N (50 lb) located in series with the actuator. In addition to the load-deflection response, the failure mode was observed and described.

Squash Load Tests

A number of axial load tests were conducted on short lengths of column removed from the test specimens. These were tested under concentric axial load in a 1.1 MN (250 kip) capacity concrete cylinder test machine in order to determine the axial capacity, or "squash load," of the built-up timber sections. Only the peak axial load and the mode of failure were noted.

EXPERIMENTAL RESULTS

Specimen Dimensions

All columns were visually inspected for significant flaws and for straightness. No flaws or significant out-of-straightness was noted in the hollow sections. The solid 4 × 4 specimens exhibited some twist and out-of-straightness, as noted in the following sections. In addition, no discontinuities of the adhesive joints were evident on visual inspection.

Overall, the column specimens are noted to be dimensionally stable. The measured column sectional dimensions varied no more than 3 mm (0.1 in.) from the assumed nominal dimensions given in Table 1. It is noted that the 4 × 4 solid specimens had not been finished after having been fabricated. All hollow specimens were tested in their finished form.

Axial Load and Deflection Results

Table 2 summarizes the applied loads and corresponding deflections recorded for each test. The following data are presented:

- *Initial buckling load*: load at which lateral deflections, Δ_x and/or Δ_y, were observed to increase without the application of further axial load and the corresponding *axial deflection*.
- *Ultimate observed load*: maximum load the column was observed to carry or the peak load that may be safely applied using the test setup and the corresponding *axial deflection* and axis in which *elastic buckling* was observed.

It is noted that no specimens exhibited or were predicted to exhibit inelastic buckling.

Representative axial load versus axial and lateral deflection responses are plotted in Fig. 3. Axial deflections represent axial shortening of the column. Lateral deflections have been plotted as absolute deflections. Lateral deflections in the x-direction correspond to deflections about the y-y axis, and vice versa. The value of P shown on each plot corresponds to the maximum allowable axial load that may be applied to the column according to the NDS equations, using $K = 0.8$. These values are also summarized in Table 2.

Discussion of Axial Load Test Results

All of the axial test specimens behaved as expected. Because of limitations of the test setup and safety concerns, the 6 × 6 and 8 × 8 specimens were not loaded to their buckling loads. These specimens, however, were loaded beyond their maximum allowable axial capacity as determined from the NDS equations.

Elastic Buckling. The 5 × 5 and 4 × 4 solid specimens were loaded through the point of elastic buckling. In all but Specimen 5 × 5-C, elastic buckling was noted in both principal directions with the final ultimate state being governed by buckling about their x-x axes.

As noted above, the larger 6 × 6 and 8 × 8 specimens were not loaded to their buckling loads. Their prebuckling lateral response may be attributed to initial out-of-straightness and the small eccentricities inherent in the test setup and the timbers themselves. Lateral response was approximately equal in both principal directions.

The 4 × 4 solid specimens had notable cambers about their y-y axes. This is plotted in Fig. 3 and results in a larger initial eccentricity. Both specimens 4 × 4-A and B exhibited elastic buckling about their y-y axes because of this eccentricity. Buckling was also evident about the x-x axis of each specimen. Ultimate response was governed by y-y axis buckling and, despite the initial eccentricity, occurred at load levels greater than the maximum allowable axial load.

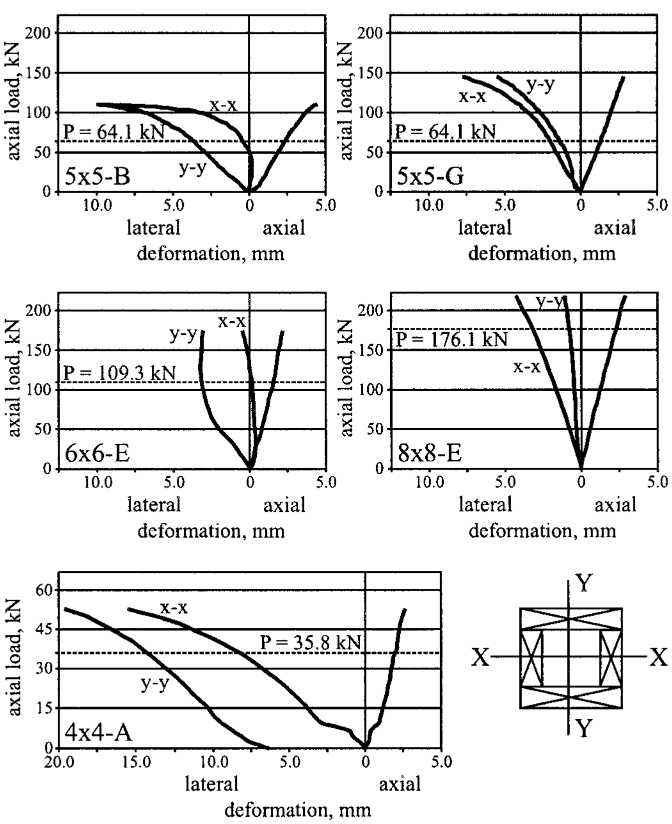

FIG. 3. Representative Axial Load versus Lateral Deflection Responses of Built-Up Specimens

Since all failures observed were due to elastic buckling, no damage to the timbers or adhesive was noted in any case. No inelastic behavior was observed.

Axial Capacity. Table 2 summarizes the axial overstrength observed in the test specimens. The overstrength ratios are ratios of the maximum observed loads to the NDS allowable loads.

Observed overstrength of the buckled specimens was approximately 1.7. It is noted that the test conditions are representative of the end conditions a timber column may be subjected to in practice. Additionally, it is likely that the columns will be designed as pin ended, using an effective length factor of $K = 1.0$. In this practical case, the observed overstrength was approximately 2.5. It is noted that the tests represented the case where K is approximately equal to 0.8. It is anticipated that these results would be consistent for the 6 × 6 and 8 × 8 specimens.

Axial Deflections. The average axial deflections measured over the column length were consistent at both initial and ultimate buckling conditions. The average axial shortening at buckling for columns having a height of 3,048 mm (120 in.) is approximately 0.065% of the column length. The axial shortening at ultimate capacity is approximately 0.1%. These values are consistent with the values of Young's modulus prescribed by the NDS Specification.

Flexural Load and Deflection Results

Table 3 summarizes the applied loads and corresponding deflections recorded for each test. The following data are presented:

- *Ultimate applied shear* and corresponding *ultimate moment*: applied load at which column failed and the corresponding *midspan deflection*

The applied midspan load versus midspan deflection responses of the columns are shown in Fig. 4. The maximum allowable load based on both flexure and shear capacities of the sections are shown in these figures as F and V, respectively. With the exception of Specimen 4 × 4-E, all tests were conducted using a 2,438 mm (96 in.) span. The axis of principal flexure, either x-x or y-y, is given in Table 3. Fig. 5 shows some representative flexural failures observed in the experimental program.

Discussion of Flexural Test Results

All specimens generally behaved as predicted. As noted, the ultimate behavior of some specimens (5 × 5-J, 6 × 6-D, 8 × 8-D, and 4 × 4-C) was clearly governed by discontinuities or knots located in regions of high stress. Only Specimen 4 × 4-C exhibited a significant variance from its predicted behavior. This is discussed below.

Flexural Failures. With one exception (6 × 6-B), flexural failures initiated on the tension face of the member and propagated toward the compression zone until the load-carrying capacity was lost. These failures are relatively brittle. Little softening of the force-deflection curve is seen prior to failure.

Orientation of the member appeared to have little effect on the flexural strength of the members. It is noted, however, that flexural failures about the y-y axis often only affected one of the vertical web timbers. However, orientation did affect the mode of failure. When oriented in the x-x direction, a flexural failure propagates as a tensile failure of the bottom flange timber. When this failure reaches the glue line, the failure propagates horizontally near the glue line as a tensile failure of the timber perpendicular to the grain. It is noted that in all cases it is the timber, rather than the adhesive, that fails in tension.

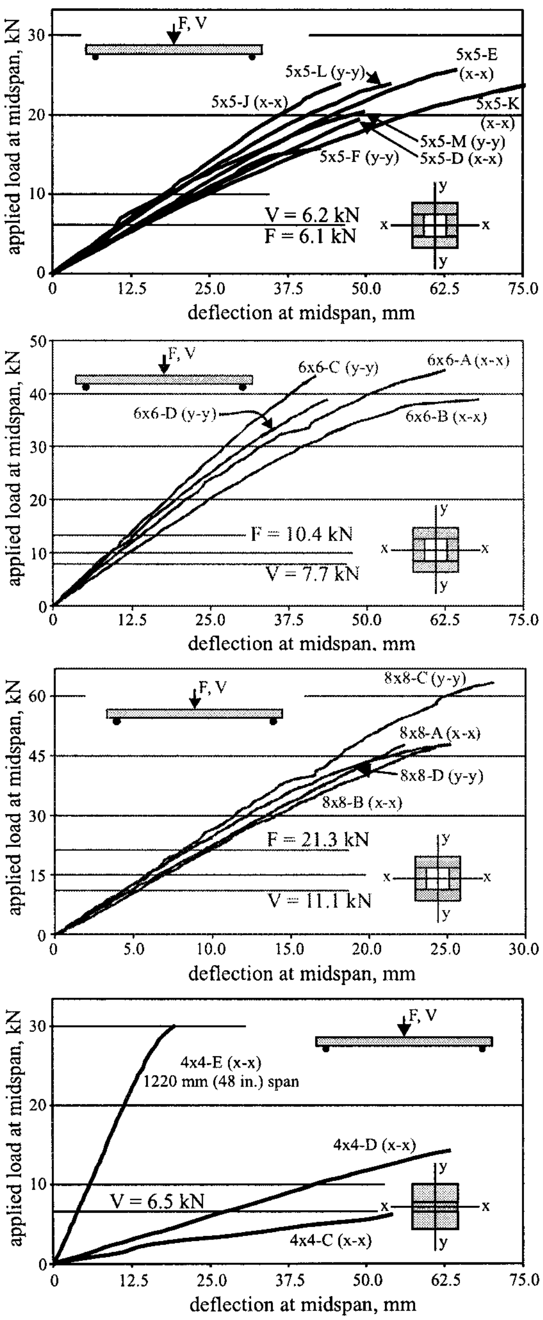

FIG. 4. Midspan Load versus Deflection Responses of Flexural Test Specimens

This type of failure is clearly shown in the close-up view of 5 × 5-J shown in Fig. 5(a). These failures can also be seen in Specimens 5 × 5-D, 6 × 6-A, 8 × 8-A, and 4 × 4-D.

Specimen 6 × 6-B exhibited a flexural failure of the compression flange timber accompanied by a significant amount of crushing perpendicular to the grain under the loading point. This was likely due to a local discontinuity in this timber. The effect of the crushing is also seen in the load-deflection response for 6 × 6-B as a significant softening of the response near the peak load (Fig. 4).

Shear Failures. Fewer shear failures were observed in the test program. With one exception (8 × 8-B), shear failures initiated in the full-depth web timbers of specimens loaded about their *y-y* axes. These failures propagated through the web timbers and were bounded by discontinuities in the timbers. The connection between abutting timbers appeared to have no effect on the shear failures, as is demonstrated by Specimens 6 × 6-C [Fig. 5(b)] and 8 × 8-D.

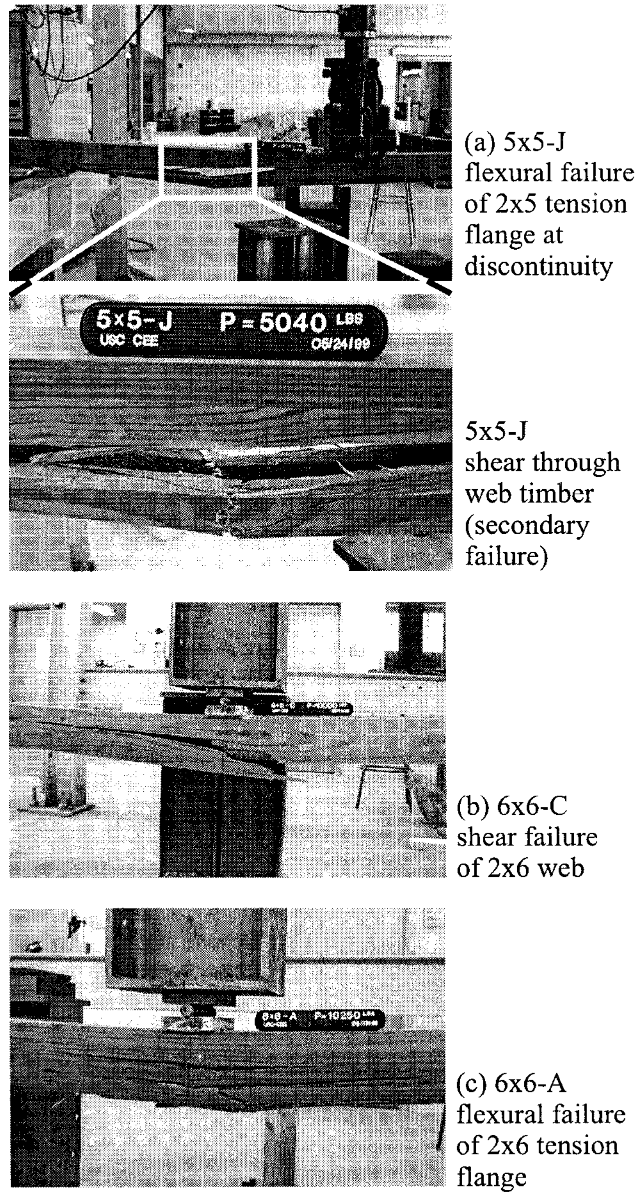

FIG. 5. Representative Flexure and Shear Failures

Specimen 8 × 8-B, tested in the *x-x* direction, exhibited a shear failure in the partial depth web timber. The 8 × 8 specimens were predicted to fail in a shear mode of behavior, thus this failure is expected.

Specimen 4 × 4-E. Specimen 4 × 4-E was tested with a decreased span (1,220 mm) in order to introduce a greater shear component to the response of this specimen. The failure of 4 × 4-E appeared to be a flexural failure of the tension timber. However, there is a clear softening of the load-deflection curve near the ultimate load shown in Fig. 4. It is suggested that this softening results from significant shear deformations which, although they did not manifest themselves in an observable crack, affected the continuity of the built-up section sufficiently to drive the observed flexural failure.

Overstrength Ratios. Table 3 summarizes the NDS predicted and observed flexure and shear loads applied to the specimens tested. The observed flexural overstrength of those specimens exhibiting a flexural failure is approximately 3.7. Similarly, the shear overstrength is approximately 2.4.

In general, the flexural overstrength gets smaller as the specimen size increases. Conversely, the shear overstrength increases with increased specimen size. This effect may result from the larger relative contribution of shear deformations

TABLE 4. Results of Squash Load Tests

Test (1)	Cut from (2)	Gross section area (mm^2) (3)	Squash load (kN) (4)	Ultimate axial stress (MPa) (5)	F_c (MPa) (6)	Overstrength (7)
			(a) 5 × 5 Specimens			
1	5 × 5-E	10,064	409	40.6	13.8[a]	3.0
2	5 × 5-D	10,064	467	46.4	33 × 109	3.4
3	5 × 5-H	10.064	474	47.1	timbers	3.4
4	sample	10,064	476	47.3		3.4
	solid 5 × 5			average = 45.4	6.6[b]	6.9
			(b) 6 × 6 Specimens			
1	6 × 6-A	13,097	614	46.9	13.8[a]	3.4
2	6 × 6-E top	13,097	767	58.6	33 × 132	4.2
3	6 × 6-E bottom	13,097	603	46.0	timbers	3.4
	solid 6 × 6			average = 50.5	6.6[b]	7.7
			(c) 8 × 8 Specimens			
1	8 × 8-D top	19,484	979	50.2	13.1[a]	4.9
2	8 × 8-B	19,484	781	40.1	33 × 180	3.1
3	8 × 8-D bottom	19,484	778	40.0	timbers	3.0
	solid 8 × 8			average = 43.4	6.6[b]	6.6
			(d) 4 × 4 Solid Specimens			
1	4 × 4-A	7,935	400	50.4	14.5[a]	3.5
2	4 × 4-C	7,935	300	37.8	33 × 89	2.6
3	4 × 4-E	7,935	405	51.0	timbers	3.5
	solid 4 × 4			average = 46.6	14.5[b]	3.2

[a]Values from NDS *Supplement* Table 4B for select structural grade Southern Pine sawn timber.
[b]Values from NDS *Supplement* Table 4D for select structural grade Southern Pine sawn timber.

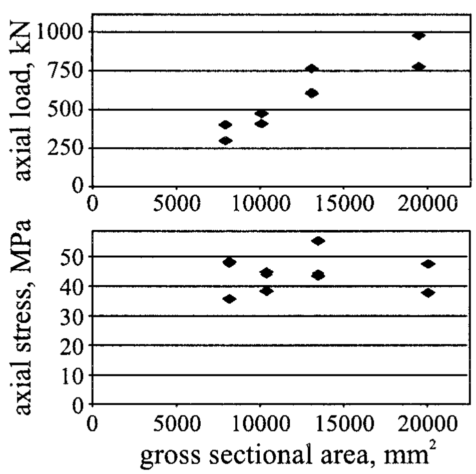

FIG. 6. Axial Load and Stress versus Section Area for Squash Load Tests

with increased specimen size. Timber is relatively weak and flexible in both shear and tension perpendicular to the grain; thus larger shear deformations result in a softening of the overall response, consequently resulting in an increase in extreme tensile stresses parallel to the grain.

Squash Load Test Results

Squash load tests were conducted on specimens cut from the ends of tested specimens. Table 4 gives the results of the squash load tests. Fig. 6 shows the squash loads and resulting axial stresses plotted against the gross section area.

The average observed squash stress was 46.3 MPa (6,720 psi). There is little evidence of the size effect typically observed in timber members. This is not surprising since the specimens tested were fabricated from more uniform smaller sections. The observed axial stress parallel to the grain for the specimens tested is given in Table 4. The table also gives values of allowable axial stress parallel to the grain, F_c, for members having the gross cross sections of the tested specimens as well as that for the components of the built-up sections.

The observed squash loads represent an overstrength factor of approximately 3.3 over the NDS allowable values for sawn timber and an overstrength factor of approximately 7.0 for solid timbers larger than 109 × 109 mm (5 × 5 in., nominally). This result clearly demonstrates an advantage of using smaller-dimensional timbers to fabricate large column sections.

Dilation resulting from the axial failure of the squash load specimens resulted almost entirely from tensile splitting perpendicular to the grain of the specimens' component timbers. The adhesive was not observed to fail in any of the specimens.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are drawn from the test program summarized in this report:

1. Axially loaded specimens having a clear height of 3,048 mm (120 in.) and an estimated effective height of 3,048 × 0.8 = 2,438 mm (96 in.) exhibited capacities of at least 1.7 times the capacities predicted by the NDS for these members. Additionally, the ultimate behavior of these columns was governed by elastic buckling.
2. Flexure-critical specimens loaded in midpoint flexure over a span of 2,438 mm (96 in.) exhibited flexural capacities approximately 3.7 times those predicted by the NDS specifications for these members.
3. Shear-critical specimens loaded in midpoint flexure over a span of 2,438 mm (96 in.) exhibited shear capacities approximately 2.4 times the capacities predicted by the NDS specifications for these members.
4. With regard to the previous three conclusions, the hollow built-up column members that were tested exhibited no discernible asymmetries. There appears to be no strong or weak direction for these column members.
5. Very short axially loaded specimens (squash load tests) exhibited capacities approximately 3.3 times greater than

the NDS specification allows. By comparison, these columns exhibited axial stress capacities approximately 7 times those of equivalent solid timber posts.

6. In no case was the Resorcinol adhesive used to fabricate the built-up sections found to fail. The adhesive appeared adequate to permit the sections to be considered as a single homogeneous section, rather than as an amalgam of smaller timbers. The following general conclusion may be drawn from the material presented in this report: *The hollow-built up sections tested may be considered "solid columns" as prescribed by Clause 3.7 of the NDS specification*.

This conclusion is drawn despite the apparent requirement of the NDS Clause 3.6.2.3 that the "individual laminations" be considered as individual solid columns.

For sections larger than 109 × 109 mm (nominally, 5 × 5 in.), the built-up columns tested have the following structural advantages over solid timbers:

1. Allowable stresses for the sawn timber components of the built-up columns may be used to determine the capacity of the member. This provides a significant increase in allowable loads for sections greater than 109 × 109 mm.
2. The sectional and longitudinal dimensional stability and clarity of the built-up columns appears superior to that of larger solid timbers.
3. The hollow provided in the built-up columns may be utilized to enhance structural connections or to provide "invisible" connections.

ACKNOWLEDGMENTS

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APPENDIX I. REFERENCES

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A = cross-sectional area;
b = column dimension;
c = distance from extreme compression fiber to neutral axis, taken as 0.80 for sawn timber;
C_D, C_M, C_t, C_F, C_i = factors reflecting load duration, wet service conditions, temperature, member size, and incision;
C_P = column stability factor;
d = column dimension;
d = smaller dimension of rectangular column section;
E = tabulated Young's modulus of timber;
f_c = allowable compression stress parallel to grain;
F = midspan load corresponding to allowable moment capacity;
F_c = tabulated allowable compressive stress parallel to grain;
F'_c = allowable design compressive stress parallel to grain;
F_{cE} = critical buckling compressive stress parallel to grain;
I = moment of inertia;
K = effective length factor;
L = column height;
L_e = effective length of column, KL;
P = allowable design axial load;
r = radius of gyration;
t = wall thickness;
V = midspan load corresponding to allowable shear capacity;
Δ_B = axial deflection of bottom platen;
Δ_B = support deflection at "bottom" of column;
Δ_M = deflection at midspan;
Δ_T = axial deflection of top platen;
Δ_T = support deflection at "top" of column;
Δ_x = lateral deflection at column midheight in plane of test setup; and
Δ_y = lateral deflection at column midheight out of plane test setup.