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Bending Properties of STP-Laminated Wood Girders

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Summary: Twenty-six foot long girders were "built-up" from dimension lumber using shear transfer plates (STPS) and metal plate connectors. The girders were tested to failure in bending to determine the efficiency of the STPS and the effect of lumber arrangement and joint location on bending strength and stiffness.

Keywords: Shear transfer plates, Lamination, Mechanical lamination, Wood girders, Lumber, Bending

Bending Properties of STP-Laminated Wood Girders

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Abstract

Twenty-seven girders were "built-up" from dimension lumber and tested to failure in bending. Each girder was 26 feet in length and contained three layers, with each of these Iayera comprised of stacked 2- by 6- and 2- by 10-inch members. Nails were used to join individual layers in ten of the assemblies. Layers in the remaining assemblies were joined with shear transfer plates (STPS). One-half of the STP-laminated girders were loaded like the nail-laminated girders, the other half were loaded differently. Test results showed that the method of laminating (nails or STPS) did not significantly affect the bending strength nor the initial bending stiffness of the girders. The direction of loading, while it did not affect initial bending stiffness, did have a significant affect on bending strength. This effect was attributed to the differences in the tensile and compressive strengths of end-joint connections.

Introduction

Mechanical-Joined Dimension Lumber (MJDL) Assemblies

In the January, 1992 issue of Fine Homebuilding (Smulski, 1992), structural engineer Christopher DeBlois gives prices for beams capable of supporting 1600 lbs. per lineal foot over a 11.5 foot clearspan. Alternatives included: solid timber, glue-laminated timber (glulam), parallel strand lumber (PSL), laminated veneer lumber (LVL), nail-laminated dimension lumber, steel 1- and flitch beams. Although he used the glulam beam for his particular application, DeBlois found the nail-laminated dimension lumber beam to be the lowest priced alternative.

What DeBlois showed in 1992, is something that post-frame building engineers have found to be true for the past two decades - when it comes to structural components, it is difficult to beat the price of mechanically-joined dimension lumber (MJDL) assemblies. MJDL assemblies include any assemblies in which mechanical fasteners (e.g., nails, bolts, screws, metal plate connectors, timber connectors, shear transfer plates, etc.) have been used to join together dimension lumber. Wtih this fairly broad definition, MJDL assemblies would include the majority of trusses fabricated from dimension lumber.

The lower cost of most MJDL assemblies can be attributed to relatively low assembly costs. For example, fabrication of MJDL assemblies does not require machinery as complex and expensive as that used to produce PSL and LVL. Also, MJDL assembly fabrication, when compared to that for glulams, PSL and LVL, requires comparatively fewer unit operations and quality control tests.

MJDL Assemblies in Post-Frame Buildings

Because of their high strength-to-cost ratio, MJDL assemblies are widely used in post-frame buildings. Three- or 4-layer laminated columns (figure 1a) are used in the vast majority of buildings, and virtually all roofs are supported with metal plate connected (MPC) trusses or stacked beams (figure 1b). In addition, built-up girders (figure 1c) are commonly used for large door headers or wherever individual trusses must be supported between columns.

The popularity of MJDL assemblies in post-frame building design can be attributed to the fact that post-frame building component selection is almost exclusively dictated by load carrying capacity and cost, and to a lesser extent by ability to resist chemical and biological agents (e.g., corrosion and decay resistance). Seldom is post-frame building component selection influenced by factors such as component size/shape, color, tire resistance, thermal conductivity, fatigue resistance, electrical conductivity, space utilization, and level of interference with plumbing, HVAC and electrical hardware.

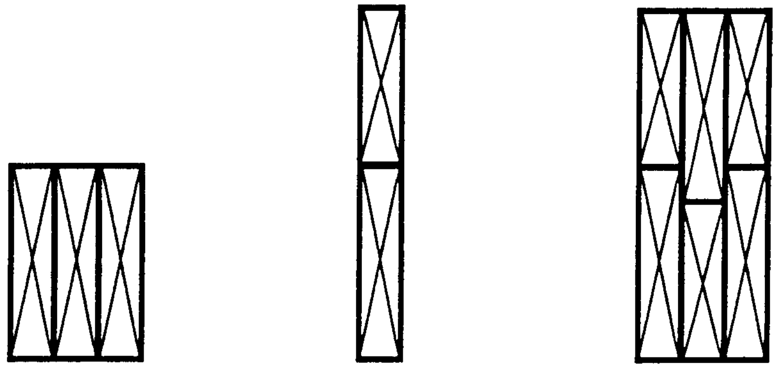


Figure 1- Cross-sections for typical (a) laminated column, (b) stacked beam, and (c) built-up girder.

Stacked Beams

Stacked beams are formed by using metal plate connectors to join the wide faces of two pieces of dimension lumber that have been stacked one upon the other. Although research on the behavior of stacked beams is limited (Percival and Comus, 1976a & 1976b), they are finding increased use as rafters in large dairy freestall barns. When properly designed and supported, stacked beams can handle considerably larger bending moments than can high grade 2- by 10-inch or 2- by 12-inch members. They are favored over MPC trusses in freestall barns because of their “clean” appearance and because they can’t be perched-on by birds.

It is not uncommon for the depth to thickness ratio of MJDL stacked beams to exceed 10. When the components are this slender, supporting the compression edge becomes crucial. In preliminary laboratory tests in which identically sized members were used to form stacked beams (figure 2a), insufficient lateral bracing resulted in a lateral shifting of the compression member relative to that of the tension member (figure 2b). This action occurred near ultimate load and resulted in some MPC tooth withdrawal.

What appears to be the ideal stacked beam is one in which a wider, lower grade material is used on the compression side of the beam, and a narrower, high grade material is used on the tension side (figure 2c). This maximizes the strength-to-cost ratio and should reduce the type of lateral shifting shown in figure 2b.

Stacked beams can be manufactured to any length by end-to-end splicing with MPCS. It is obviously best to avoid end joints in high moment regions.

When designing stacked beams, it is typically assumed that there is no slip between the stacked members. The shear force that must be resisted by the MPCS connecting the stacked members is then determined using procedures of conventional engineering mechanics. The allowable design moment capacity is calculated according to standard procedures (AF&PA, 1997) with a special stacked beam reduction of 20% typically used to account for design assumptions and allowed fabrication tolerances (Brakeman, 1998).

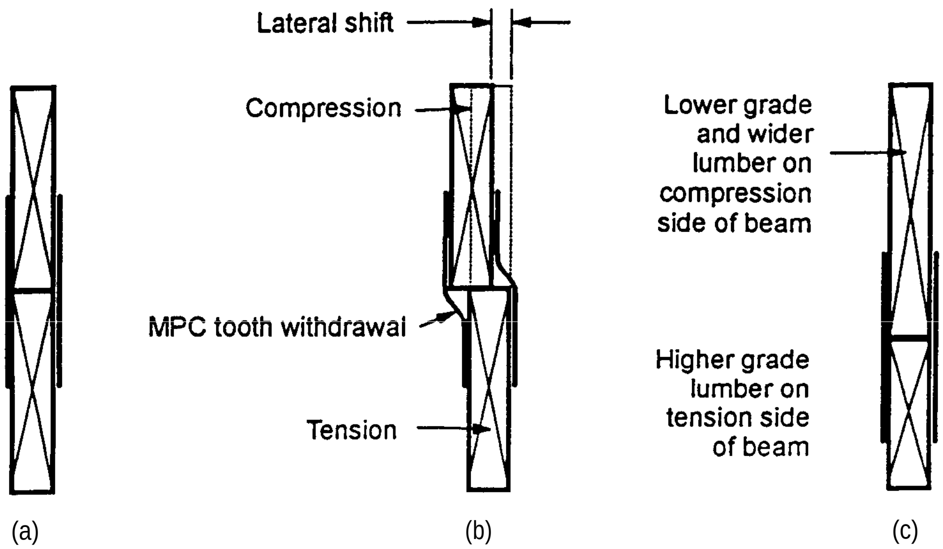


Figure 2- Stacked beams with identically sized members and insufficient lateral bracing: (a) unloaded, and (b) under high bending load. (c) Stacked beam designed to minimize cost and lateral shifting.

Built-Up Girders

When two or more stacked beams are laminated together, the resulting assembly is referred to as a built-up girder (figure 1c). The design capacity of a built-up girder is generally at least as great as the sum of the design capacities of the individual stacked beams or layers. This is because (1) laminating increases the effective width of the assembly which reduces lateral instability under load, and (2) end joints in adjacent layers can be staggered. The latter enables adjacent layers to support each other's joint regions.

The design of built-up girders is a two-step process. First, layers are treated as individual stacked beams to determine MPC size and location. Second, the location of end-joints, when present, must be established. In selecting joint location, the designer attempts to (1) stagger and adequately space joints for optimum strength, (2) keep joints out of critical areas, and (3) limit the length of individual members (generally to something less than 16 feet).

Shear Transfer Plates

Shear transfer plates (STPS) are light gauge steel plates with teeth on both sides. Figure 3 shows a plug style STP that was developed by Jack Walters and Sons Corporation of Allenton, WI and subsequently used to produce STP-laminated columns.

STPS are manufactured in a variety of sizes by stamping them from coils of thin gauge steel in a process similar to that used to produce metal plate connectors (MPCS). Once fabricated, the plates can be installed in the factory or on the jobsite using the same equipment used to install MPCS. Pressing is typically done in two stages. First, the plate is completely pressed into one of the members using a special steel pressing plate that fits over the STP. The pressing plate is then removed, and the other piece of wood is placed over the STP and pressed into place. Although a single stage process could be used to simultaneously press the plate into both wood members, it is generally not used as it puts a permanent wave in the plate (making it difficult to get a tight connection), and it requires more energy and produces weaker and more flexible connections than does the two stage pressing process (Wolfe and others, 1993).

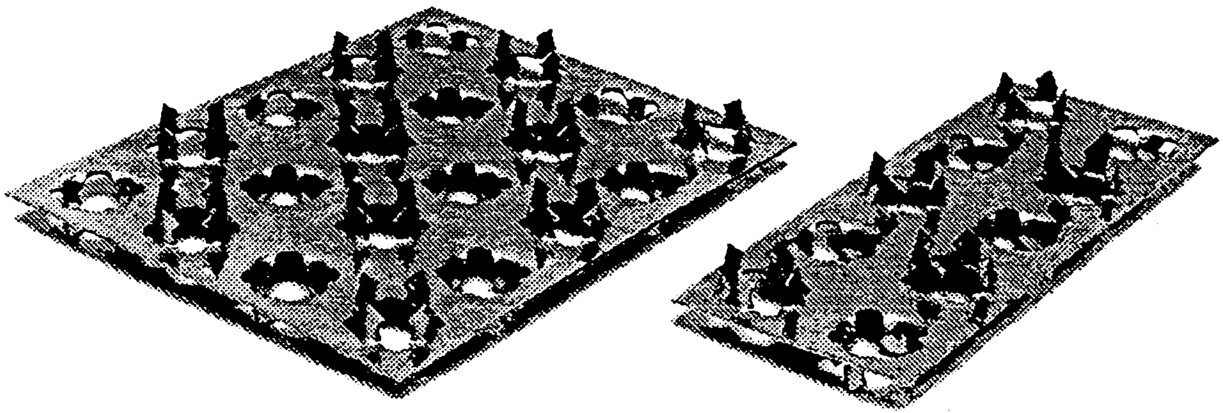


Figure 3- Shear transfer plates (STPS) with plug density of 1 plug/in^2

Although STPS are principally used to transfer shear between wood layers, they can be used like MPCS to connect two wood members that have been butted together. Using an STP in this manner is only practical when an adjacent wood member is present as shown in figure 4. The adjacent member performs two functions. First, the adjacent wood member decreases the load level at which compressive forces buckle the plate at the butt joint. Second, it decreases the amount of strain which builds up in the flat (non-tooth podion) of the plate near the butt-joint. This is due to the fact that tooth forces are not transferred along a plate but instead, are transferred through the plate, into the adjacent wood member, past the butt-joint and then back through the plate as shown in figure 4.

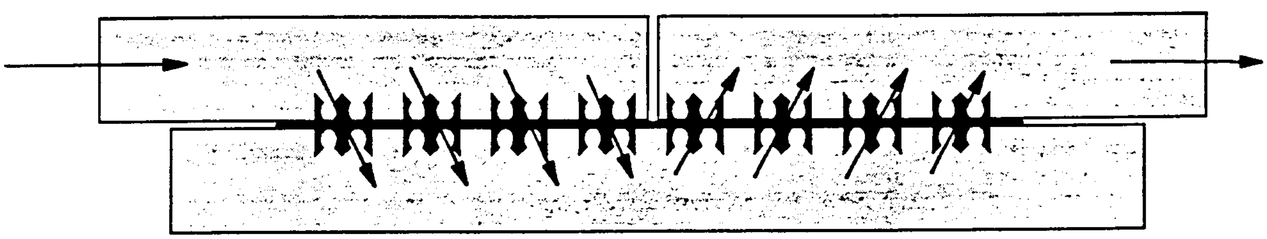
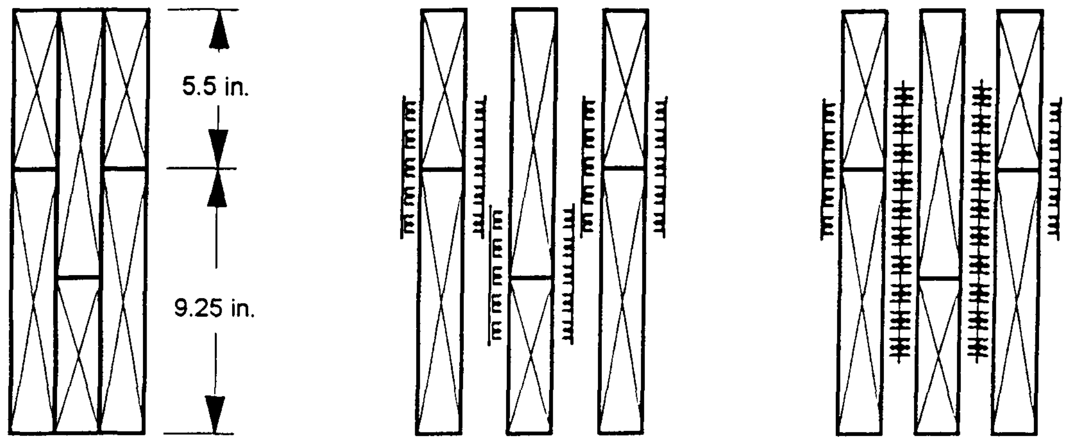


Figure 4- Load transfer around a butt joint via a shear transfer plate

<u>STP-Laminated Girder Development</u>

Research conducted in the early 1990's on STP connections (Wolfe and others, 1993) and STP-laminated columns (Bohnhoff and others, 1993;) demonstrated the shear transfer efficiency of the Jack Walters & Sons STP. This research led to the subsequent design of the STP-laminated girder (figure 5c) as a potential replacement for the nail-laminated design being used by Jack Walters & Sons (figure 5b) The advantage of the STP-laminated girder design is that STPS not only replace the nails used for laminating, but also all unexposed MPCS. This, in turn, reduces the total amount of steel required for girder assembly. Although the new STP-laminated girder design appeared sound, it was not known how its bending strength and stiffness would compare with that of a comparable nail-laminated design. In addition, it was not known to what extent bending strength and stiffness were influenced by end-joint locations, nor was it entirely clear that STP-laminated girders could be fabricated with existing equipment.



(a) (b) (c)

Figure 5- Built-up wood girder showing (a) member lay-up, (b) metal plate connectors (MPCs), and (c) inner MPCS replaced with shear-transfer plates (STPs).

<u>Research Objective and Scope</u>

The objectives of this research were to:
1. Determine the bending strength and stiffness of (1) two STP-laminated girder designs (each with a different arrangement of end-joints), and (2) a comparable nail-laminated girder design.
2. Compare the difference in the bending properties of the three girder designs.

The scope of the project was limited to one girder size, one lumber grade and a fixed density of plates.

Experimental Materials and Methods

<u>Girder Design</u>

The first step in girder design was to establish overall size. After an assessment of actual girder use and with due consideration of test machine capacity, a three- layer assembly featuring stacked 2- by 10-inch and 2- by 6-inch members was selected (figure 5a). This arrangement has an actual width of 4.5 inches and a depth of 14.75 inches. Overall girder length was fixed at 26 feet.

The second step in girder design was joint pattern selection. The first goal in this process was to select an ideal pattern - one that would maximum bending strength. After some consideration, the arrangement shown in figure 6 was selected. When viewing this pattern, it is important to keep in mind that it was designed to be loaded so that edge A would be in compression (i.e., members 1, 2, 5, 6, 10 and 11 in compression). Key elements of this design include: (1) no tension side joints within seven feet of midspan when edge A is in compression, and (2) a minimum joint spacing of three feet. Tension side joints were kept out of the midspan area by placing the longest three members in the assembly (members 3, 8, and 13) on the tension side. Because of concern that member 8 would carry a disproportionate amount of load if it was a 2- by 10-inch member (this since its ends are furthest from girder midspan), member 8 was selected to be a 2- by 6-inch member. This assignment determined the size of all the remaining 12 members. Lastly, it should be noted that 20-foot dimension lumber had been secured for this study prior to girder design, and to avoid material waste, a pattern was selected that would best utilize the 20-foot stock.

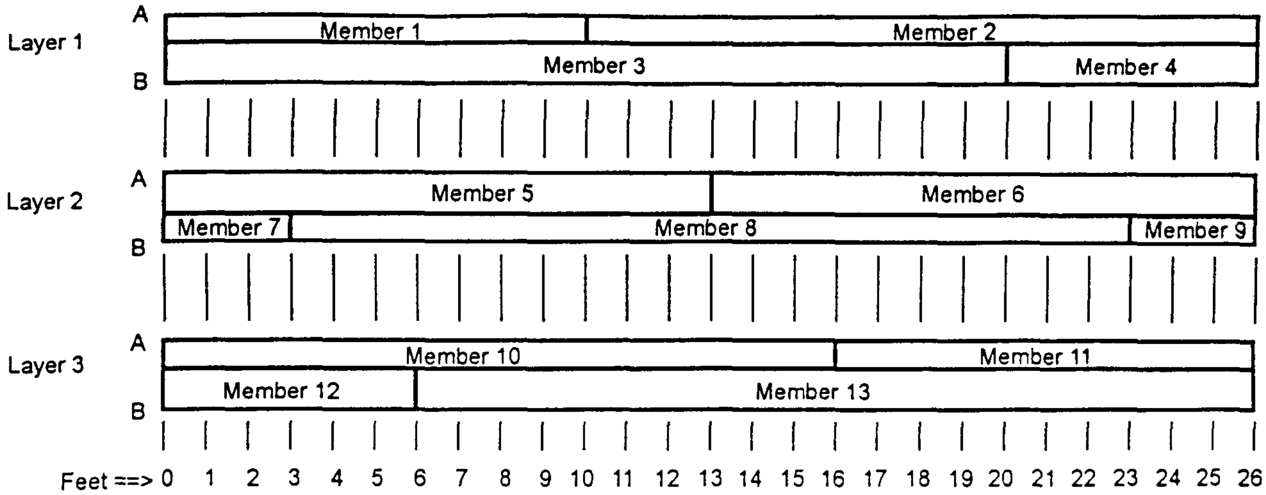


Figure 6- End-joint location for test assemblies

The second goal in joint pattern was to select a *less than ideal* pattern. Initially the thought was to create a design with 50 to 100 percent more end-joints than the pattern in figure 6. However, atler considering that actual joint location was likely to be just as important as number of joints, it was decided to double the number of STP-laminated girders fabricated with the pattern in figure 6, but load one-half of them so edge A was in compression, and load the other half so edge B was in compression. When edge B is in compression, there are three tension side end-joints within 4 feet of midspan. On the assumption that tension side end-joints initiate failures, it was hypothesized that reverse loading of the ideal pattern would be associated with decreased bending strength. Validating this hypothesis was felt to be important as it would demonstrate the need to identify the "up" side of girders for field placement.

The final step in the design process was to determine the size and location of all mechanical fasteners. For the experimental nail-laminated girder, a Jack Walters & Sons production design was essentially copied. As figure 7 shows, stacked members were plated together using 6- by 10-inch 20-gage MPCs with an on-center spacing of 2 feet. This pattern was duplicated on each side of each layer. In addition, a 14- by 9-inch 16-gage MPC was embedded into each side of each end-joint. Individual Iayers were connected with 0.131- by 2.75-inch pneumatically driven nails spaced every 6 inches on each side of the assembly.

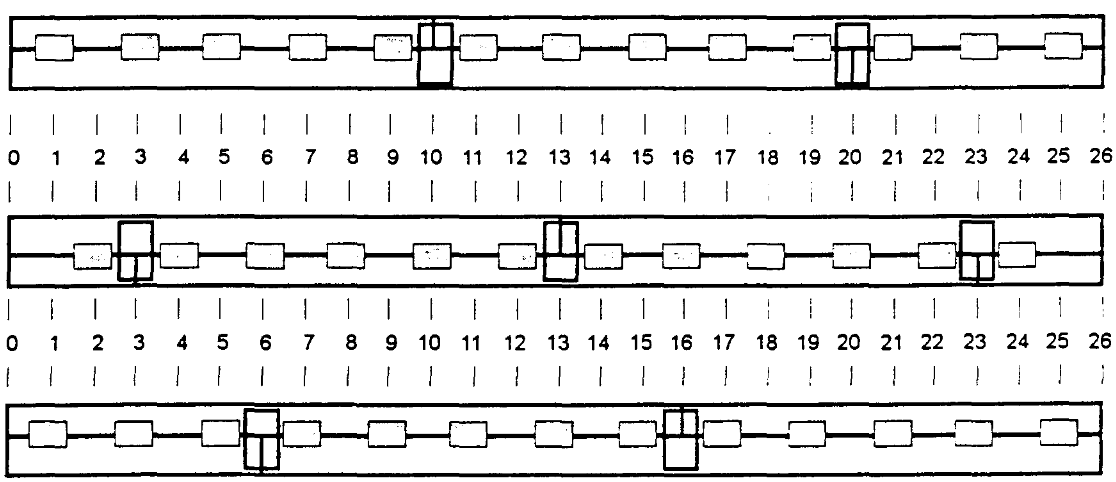


Figure 7- Metal plate connectors location in nail-laminated girders. Same pattern both sides.

Plate locations for the experimental STP-laminated girders are shown in figure 8. By placing 5-by 10-inch 20-gage STPS vetiically with an on-center spacing of one foot, the amount of steel in shear at each edge joint was approximately the same as that for the nail-laminated girder design (figure 7).

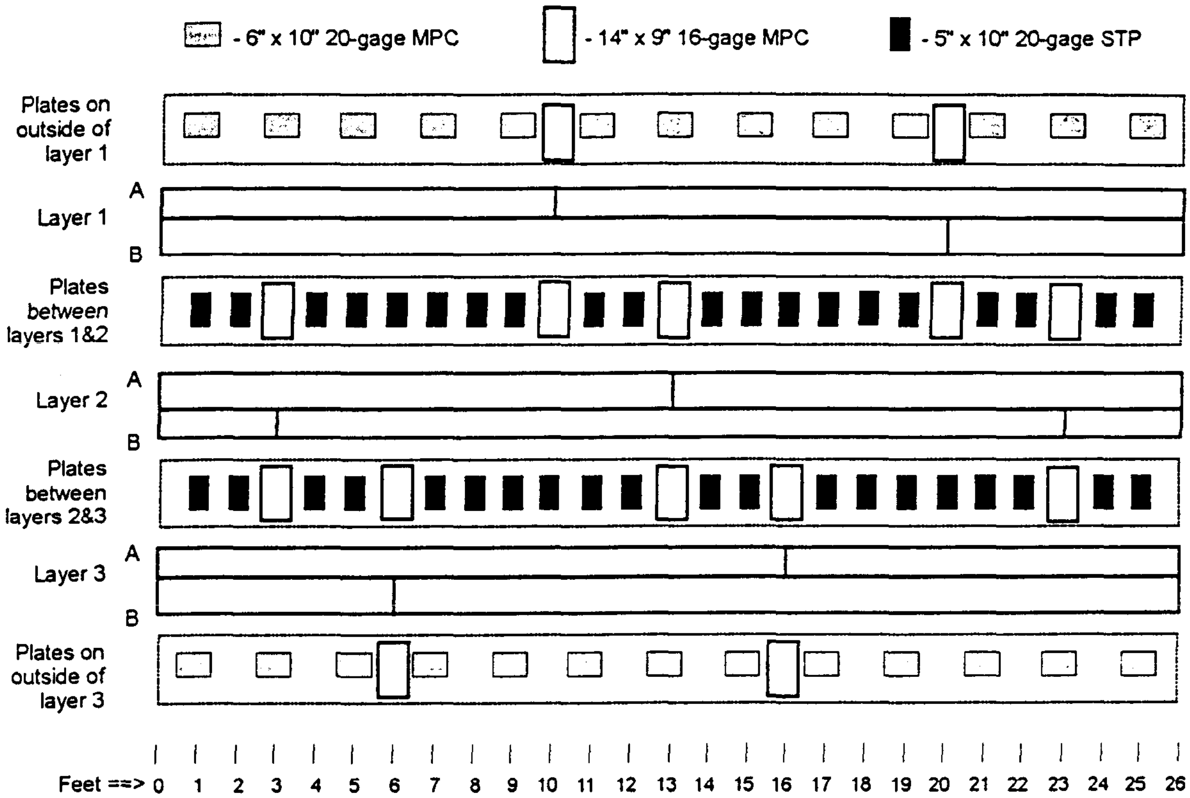


Figure 8- Location of shear transfer plates and metal plate connectors in STP-laminated test assemblies.

Throughout the remainder of the paper, the three different girder test assemblies are identified as follows:

1. Design NAIL-A: Nail-laminated assembly loaded so that edge A is in compression.
2. Design STP-A: STP-laminated assembly loaded so that edge A is in compression.
3. Design STP-B: STP-laminated assembly loaded so that edge B is in compression.

Lumber Preparation and Allocation

One hundred thirty-eight pieces of 20-foot 2- by 6-inch lumber and an equal number of 20-foot 2- by 10-inch lumber were obtained for this study. This was enough lumber to build 10 replications of each design. All lumber was machine stress rated surfaced-dried (KD-19) Southern Yellow Pine. The 2- by 6-inch lumber was grade-stamped 2400f-2.0E. The 2- by 10-inch lumber was grade-stamped 2250f-1.9E.

The lumber was stored inside a Jack Walters & Sons' manufacturing facility for approximately one year. Each piece was then given an identification number, measured at three locations for moisture content, and weighed. The modulus of elasticity (MOE) of each piece was then determined using a flatwise vibration technique.

To begin the allocation process, 120 members were randomly selected from each group of 138. Next, 60 of the 2- by 10-inch pieces were randomly selected and each cut into 13-, 6- and l-foot pieces with the 1-

foot pieces being discarded. Similarty, 60 of the 2- by 6-inch pieces were randomly selected and each cut into 16-, 3- and 1-foot Pieces (with the 1-foot pieces also being discarded), and 30 more 2- by 6-inch pieces were randomly selected and each cut into two 10-ft pieces. This cutting Ieft several groups with sixty identically sized members per group. The lumber in each of these groups was ranked by MOE, and then divided into subgroups of three such that the stiffest three pieces were in the same subgroup, the next stiffest three pieces in the next subgroup, etc. The three members in each of these subgroups were then allocated as follows:

1. A replicate number between 1 and 10 was randomly selected.
2. A coin flip was used to select one of the two member numbers associated with the length being allocated. For example, member numbers 5 and 6 are associated with 13-foot 2- by 10-inch pieces (figure 6).
3. If the combination of the replicate number (from step 1) and member number (from step 2) had not previously been selected, the two numbers were marked on each of the three pieces in the subgroup and then randomly assigned to the three girder designs.

With the proceeding allocation process, 10 matched sets of 3 were created for the three different girder test assemblies, ensuring very similar distributions of lumber MOE among the three different designs.

Girder Fabrication

The nail-laminated girders were assembled in a two step process. First, individual stacked beams were fabricated using conventional truss fabrication equipment, then the individual stacked beams were laminated using a hand-held pneumatically nailer. No special fixturing or clamping were used during this assembly process.

STP-laminated girders were assembled in a three step process. First, conventional truss fabrication equipment was used to press in all MPCs. Next, a large press brake was used to simultaneously press all STPS into the middle girder layer (figure 9). During this operation the STPS were fixtured-in-place above and below the wood layer by thick steel plates with holes that accommodated the plate plugs. Because the layer was longer than the press brake, STPS were first pressed into one end of the layer, the layer was shifted down the press brake and the remaining STPS were pressed into place. In the third step of the assembly, the outside layers were tacked onto the sides of the middle layer and the press brake was used to seat the STPS in the outer layers. Again, because of the length of the assembly, Only one end of the girder could be pressed at a time.

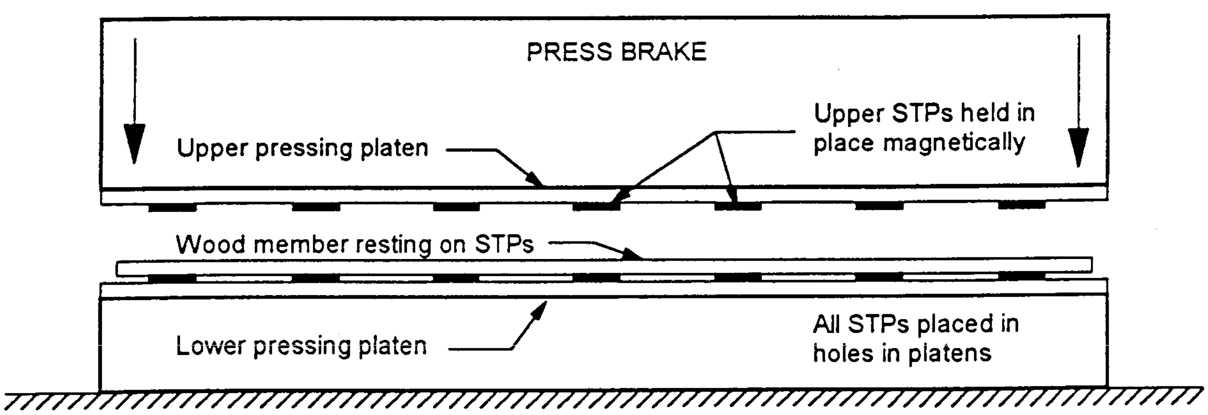


Figure 9- Use of press brake to install plates in center layer of STP-laminated girder.

After girder fabrication was completed, it was discovered that 3 of the assemblies had been incorrectly assembled. Specifically, during the first step of STP-laminated girder fabrication, the MPCs for replications 2, 3 and 4 of design STP-A, were pressed into the wrong side of layer 1.

Testing Procedure

Girders were transported to the Biological Systems Engineering (BSE) Structural Testing Laboratory at the University of Wisconsin-Madison, stored, and tested approximately one year after fabrication.

Bending tests were conducted in accordance with ASTM D 198 (ASTM, 1992) where applicable. Load was applied at one-third points - a common load arrangement for testing and a loading common to many field installed girders. The load-head rate was fixed at 0.40 in/min (10 mm/min) for all tests. The location of the load points, support reactions, and points of lateral support are shown in figure 10. To measure deflections, a spring-tensioned wire was drawn between nails driven at girder mid-height at locations directly above the supports. The relative displacement between the wire and the girder at load-points were measured by fastening linear variable differential transformers (LVDTs) to the girder at the load-points and hooking the LVDT cores to the wire. To avoid damage to the LVDTs, they were removed once the load-point deflections reached 2 inches. A computer-based data acquisition system was used to record load-point deflections and load data at 0.5 second intervals. Wood moisture content was checked at the time of test with a resistance type moisture meter.

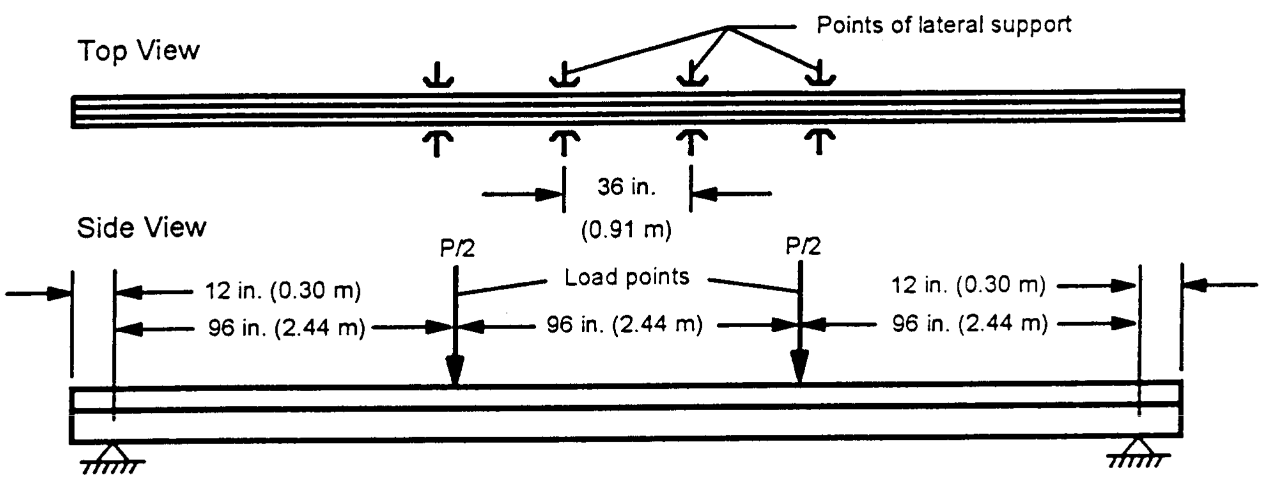


Figure 10- Location of load points, support reactions, and points of lateral support for 3-layer built-up wood girder tests.

Results

Lumber Properties

Lumber properties are compiled in Table 1. This table lists mean values and corresponding coefficients of variation for both dynamic MOE and specific gravity. The moisture content of the lumber at the time of fabrication averaged 13.2%. At test time the average moisture content was 11.1%

Table 1 - Lumber Properties

Nominal lumber size (in. x in.)	Number of pieces	Modulus of elasticity*		Specific Gravity**	
		Mean ($\times 10^6$ lb/in^2)	COV (percent)	Mean	COV (percent)
2 x 6	120	2.21	12.9	0.53	8.3
2 x 10	120	2.34	10.5	0.57	7.2

* Determined by a flatwise vibration technique

** Based on calculated oven-dry weight and nominal dimensions

Girder Properties

Initial bending stiffness and ultimate midspan bending moments for the three different girder designs are compiled in Table 2. Values are presented in terms of stiffness and bending moment rather than MOE and modulus of rupture (MOR) since the latter have no direct physical meaning because of the complex stress distributions in the assemblies.

Initial bending stiffness was defined as the slope of the load versus average load-point deflection curve between total loads of 1000 and 7000 Ibs. This 6000 lb. range was selected after an examination of the data showed all load-displacement curves to be very linear over this range. The actual stiffness values in Table 2 were obtained by linear least squares regression. The lowest R-squared value associated with these regression analyses was 0.999.

Table 2- Girder Initial Stiffness and Ultimate Midspan Bending Moment

Replicate number	Initial stiffness* (lbs./in.)			Ultimate midspan bending moment ($\times 10^3$ in.-lbs.)		
	NAIL-A	STP-A	STP-B	NAIL-A	STP-A	STP-B
1	6590	6300	6110	1090	1231	933
2	6440	**	6690	1109	**	951
3	6410	**	6420	1068	**	1031
4	6260	**	6190	1172	**	907
5	6490	6800	6850	1182	1083	950
6	6610	6800	7190	1179	1328	904
7	6270	5990	6230	1094	1205	899
8	6750	6360	6290	1231	1291	892
9	6590	6430	6690	1047	1008	881
10	6310	6860	6650	994	1147	932
Mean	6470	6510	6530	1120	1180	930
COV	2.5%	5.0%	5.2%	6.5%	9.6%	4.7%

* Slope of total load versus average load-point deflection curve between total loads of 1000 and 7000 lbs.

** Girder incorrectly fabricated and omitted from analysis.

Failure Modes

After all tests were completed, each assembly was delaminated and a sketch made of plate and wood failure locations. This information is summarized in Table 3. Common failures in addition to wood failures included: (1) MPC failure at tension side joints, and (2) shear of MPCS along the edge between 2- by 6- and 2- by 10-inch members. MPC failures at the tension side joints were due to fracture of plate strands, tooth withdrawal, wood shear, or a combination of these three failures. It is important to note that there is no mention of STP withdrawal. This does not mean STP withdrawal did not occur it just could not be clearly identified after assembly delamination. More than likely there was some withdrawal of the STPS that were embedded on the opposite side of the surface containing the MPCs that failed in shear.

The numbers in Table 3 represent the number of times the failure occurred in the assembly. For example, a 3 under the category of MPC *failure - tension side joint* means that the failure appeared at three different tension side joints within that particular girder. Similarly, a 2 under the category of *wood failure- tension side 2x10* means that 2 different 2- by 10-inch members on the tension side of the assembly showed one or more wood failures. Note that no attempt was made to distinguish between different types of wood failures because the complex distribution of load within the assembly made it difficult to distinguish between such failures as horizontal shear and tension perendicular-to-grain. Also, no attempt was made to distinguish between initial and secondary failures because of (1) the number of simultaneously appearing failures, and (2) the inability to identify when failures occurred in the middle layer.

Table 3- Failure Location and Frequency*

Failure Description	Replicate Number										Avg.
	1	2	3	4	5	6	7	8	9	10	
	Design NAIL-A										
MPC failure - tension side joint	2	1	1	1	-	-	1	-	1	1	0.8
MPC shear between stacked members	-	1	-	-	-	-	-	-	-	-	0.1
Wood failure – tension side 2x10	1	1	2	1	1	2		1	2	3	1.4
Wood failure – compression side 2x10	-	1	-	-	1	-	-	1	-	-	0.3
Wood failure – tension side 2x6	-	-	-	-	-	-	1	1	-	1	0.3
Wood failure – compression side 2x6	1	-	-	-	1	-	1	-	-	-	0.3
	Design STP-A										
MPC failure - tension side joint	-	**	**	**	2	1	1	-	2	-	0.9
MPC shear between stacked members	-	**	**	**	-	-	-	-	-	-	0
Wood failure – tension side 2x10	2	**	**	**	1	1	1	1	1	2	1.3
Wood failure – compression side 2x10	-	**	**	**	-	-	1	-	1	-	0.3
Wood failure – tension side 2x6	-	**	**	**	-	1	-	-	1	-	0.3
Wood failure – compression side 2x6	-	**	**	**	-	-	1	1	-	-	0.3
	Design STP-B										
MPC failure - tension side joint	3	2	3	3	3	3	1	1	2	2	2.3
MPC shear between stacked members	-	-	1	2	-	-	-	-	1	-	0.4
Wood failure – tension side 2x10	-	1	-	1	-	-	1	-	1	-	0.4
Wood failure – compression side 2x10	-	-	-	-	2	2	-	1	-	1	0.6
Wood failure – tension side 2x6	-	-	-	-	1	-	1	1	1	1	0.5
Wood failure – compression side 2x6	-	-	1	-	-	-	-	-	-	-	0.1

* Numbers in table indicate number of members or number of joints exhibiting same failure.

** Girder incorrectly fabricated and omitted from analysis.

Discussion

Lumber Properties

The dynamic MOE values measured at the time of girder fabrication exceeded NDS values. The average dynamic MOE of 2.21x10^6 lb/in.^{2} for the 2- by 6-inch lumber exceeded the grademark value of 2.0x10^6 lb/in.^{2} by 10.5%. The dynamic MOE of 2.34x10^6 lb/in.^{2} for the 2- by 10-inch lumber exceeded the grademark value of 1.9x10^6 lb/in.^{2} by 23%.

Bending Stiffness

The mean initial bending stiffness of girder designs NAIL-A, STP-A and STP-B were calculated to be 6470, 6510, and 6530 Ibs./in, respectively. Comparison testing at the 0.05 level showed that there was no significant difference between the three mean initial stiffness values. This finding was not unexpected. The only difference between designs NAIL-A and STP-A was in the method of lamination - a design variable that only influences built-up girder strength and stiffness when a good percentage of the applied load is being transferred between individual stacked beams. In this study, the three layers were of similar stiffness and all were forced by the load-head to displace the same amount. Consequently, interlayer shear transfer forces were low, making method of lamination a non-factor in determining assembly strength and stiffness.

The lack of a significant difference between the stiffness of the STP-laminated girders loaded on edge A and those loaded on edge B was not unexpected. The load-slip behavior of a MPC connection at low loads is generally the same regardless of whether the joint is in compression or tension. Consequently, one would not expect to see a difference in the stiffness of designs STP-A and STP-B when both are under low load loads.

Averaging the three mean initial stiffness values yields a stiffness value of 6500 Ibs./in. If the girders were assumed to be homogeneous solids 4.5 inches thick and 14.75 inches deep, this stiffness value would be associated with an apparent *edgewise bending* MOE of 1.99x10^6 ibs/in^2. This is about 12.5% less than the average dynamic MOE of the lumber as determined by flatwise vibration. This difference can be attributed to lack of complete composite action in built-up girders, and to the fact that dynamic MOE values determined by flatwise vibration generally over-estimate apparent edgewise bending MOES.

Bending Strength

The mean ultimate midspan bending moments for girder designs NAIL-A, STP-A and STP-B were calculated to be 1.12, 1.18, and 0.93 million in.-lbs., respectively. Comparison testing at the 0.05 level showed that there was no significant difference between the bending strengths of designs NAIL-A and STP-A, but that the strength of design STP-B was significantly less than the for both designs NAIL-A and STP-A.

The lack of a significant difference between the ultimate bending strengths of designs NAIL-A and STP-A is likely due to low interlayer shear transfer forces. As previously explained, low interlayer shear forces occur when individual layers (1) have similar bending stiffnesses, and (2) are forced by a load distributing element to displace the same amount. When intertayer shear forces are low, variations in mechanical lamination are unlikely to have a significant impact on assembly strength.

The similarity in the ultimate strengths of designs NAIL-A and STP-A is reflected in the similarity of the location and frequency of failures for the two designs. With regard to designs NAIL-A and STP-A, the most frequently occurring failure was a wood failure in a tension side 2x10. The second most common failure was MPC failure at one or both of the tension side end-joints located seven feet from midspan. There was no MPC failure at compression side end-joints. This is not surprising as MPC plated end-joints can generally handle greater compressive forces than tension forces due to lumber end-bearing contributions.

As expected, loading the girders such that edge B was in compression instead of edge A resulted in a significant reduction in ultimate bending strength due to connection failures at tension side end-joints. As designed, there were three end-joints in the constant bending moment region (i.e., the center 8 feet) of each girder. The reversal of load from edge A to edge B (figure 6) resulted in these end-joints being subjected to tensile forces instead of compressive forces. In five (or one-half) of the STP-B assemblies, MPC failure occurred at all three of these tension side end-joints. In three of the other five STP-B assemblies, failures occurred at two of the three tension side end-joints, with only one of the joints failing in each of the remaining two STP-B assemblies.

Bending strength design values are generally calculated by dividing the flfth percentile estimate of ultimate bending strength by a factor of 2.1. The 2.1 value is the product of a 1.3 factor of safety and a 1.6 load duration factor. Fifth percentile estimates associated with three different distribution types are given in Table 4. Dividing the average of these point estimates by 2.1 results in allowable design bending moments of 472000, 475000, and 401 000 in-lbs. for designs NAIL-A, STP-A and STP-B, respectively.

Table 4 - Fifth Percentile Estimates of Ultimate Midspan Bending Moment

Distribution	Bending Moment (x 10^3 in-lbs.)		
	Design NAIL-A	Design STP-A	Design STP-B
Normal	997	998	856
Lognormal	1,001	1,004	860
2-Parameter Weibull	975	990	813
Average	991	997	843

For comparative purposes, the bending moment for a complex mechanically-laminated assembly is often converted to an *effective* bending stress by treating the assembly as a homogeneous solid of equivalent size and shape. For the girders tested in this study, an equivalent homogeneous solid would have a section modulus of 163.2 cubic inches. Dividing this value into the previous allowable design bending moments yields allowable effective design bending stresses of 2890, 2910 and 2460 lb/in.^{2} for designs NAIL-A, STP-A and STP-B, respectively. All three of these values exceed the grademarked values of 2400 and 2250 lb/in.^{2} appearing on the 2-by 6- and 2- by 10-inch lumber, respectively. However, when the 2400 and 2250 values are increased 15% (to 2760 and 2590 lb/in.^{2}) for repetitive member use, they both exceed the 2460 lb/in.^{2} value associated with Design STP-B. Other points to consider in these comparisons are: (1) past research has shown the NDS repetitive member factor to be low for unspliced mechanically-laminated dimension lumber, and (2) based on its relatively high dynamic MOE, the lumber used in this study was more characteristic of grades higher than that appearing on the grademark. Unfortunately, because individual members were not tested in this study, the true strength of the lumber could not be ascertained.

Finally, it is impottant to note that **the effective design bending stresses calculated for the built-up girders should not to be used in design.** This is because the assemblies in this study were: (1) insufficient in number to accurately estimate fifth percentile values, and (2) all fabricated from the same two batches of lumber - batches that do not appear to be representative of their respective grades.

Summary

Twenty-seven built-up girders were tested to failure in bending. Each girder was 26 feet in length and contained three layers, with each of these layers comprised of stacked 2- by 6- and 2- by 10-inch members. Nails were used to join individual layers in ten of the assemblies. Layers in the remaining assemblies were joined with shear transfer plates (STPS). Of the STP-laminated girders, one-half were loaded on the same edge as the nail-laminated girders, the other half were loaded on the opposite edge. Test results showed:

1. Neither the method of lamination nor the direction of loading had a significant effect on the initial bending stiffness of the built-up girders.

2. The displacement of the built-up girders was about 12% greater than would have been predicted by assuming that each assembly was a homogeneous solid with an edgewise bending MOE equal to the dynamic MOE measured for the lumber at time of fabrication. This decrease in stiffness was partly attributed to a lack of complete composite action between stacked members.

3. The method of lamination did not significantly affect the bending strength of the built-up girders.

4. Load direction had a significant effect on built-up girder bending strength. Mean strength was reduced approximately 22% when assemblies were loaded on their opposite edge. This load reversal placed end-joints within the constant moment region of the assemblies in tension resulting in MPC failure at lower assembly loads.

5. The effective allowable design bending stress for assemblies with critical end-joints located in compression regions was calculated to be 2900 lbs./in.^{2}. This suggests that high strengths can be obtained by proper design of built-up girders. Note: because of uncertainties associated with limited testing, the 2900 lbs./in.^{2} value should not be used in actual design.

References

AF&PA. 1997. National design specifications for wood construction. American Forest & Paper Association. Washington D.C.

ASTM. 1992. Standard methods of static testing of timbers in structural sizes. ASTM D 198-84. Annual book of ASTM Standards, Vol. 0409. Philadelphia, PA: American Society for Testing and Materials.

Bohnhoff, D. R., A. B. Senouci, R. C. Moody, and P.A. Boor. 1993. Bending properties of STP-laminated posts. Presented at the 1993 International ASAE Summer Meeting. Paper No. 934060. ASAE, 2950 Niles Rd., St. Joseph, MI 49085

Brakeman. D. 1998. Personal communication. June 19, 1998.

Percival, D. A. and Q. B. Comus. 1976a. *Report No. 3 Full Scale Tests of a 16.5-foot x 14.5-inch Plated Beam.* Prepared for Lumbermate Company, St. Louis, Missouri. August, 1976.

Percival, D. A. and Q. B. Comus. 1976b. *Report No. 4 Full Scale Tests of a 16.5-foot x 16.5-inch Plated Beam.* Prepared for Lumbermate Company, St. Louis, Missouri. September, 1976.

Smulski, S. 1992. "Glue-laminated timbers." *Fine Homebuilding.* December 1991/January 1992, pp. 55-59.

Wolfe, R. W., D. R. Bohnhoff, and R. Nagel. 1993. Stiffness and strength properties of shear transfer plate connections. Res. Pap. FPL-RP-517. Madison, WI. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 25 p.