BENDING PROPERTIES OF LAMINATED-LUMBER GIRDERS

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ABSTRACT. *Twenty-seven girders were "built up" from dimension lumber and tested to failure in bending. Each girder was 7.9 m (26 ft) in length and contained three layers, with each of these layers comprised of stacked 38 × 140 and 38 × 235 mm (nominal 2 × 6 and 2 × 10 in.) members. Nails were used to join individual layers in 10 of the assemblies. Layers in the remaining assemblies were joined with shear transfer plates (STPs). One-half of the STP-laminated girders and all of the nail-laminated girders were loaded such that the "top" of the girder (a.k.a. edge A) was in compression. The other half of the STP girders were turned upside-down and loaded such that the bottom of the girder (a.k.a. edge B) was in compression. 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. Changing the surface of load application from edge A to edge B did not affect initial bending stiffness, but 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.*
Keywords. *Lamination, Laminated lumber, Mechanical lamination, Wood girders, Built-up girders, Shear transfer plates, Lumber, Bending, Wood design.*

Mechanically joined dimension lumber (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. With this fairly broad definition, MJDL assemblies would include the majority of trusses fabricated from dimension lumber.

Because of their high strength-to-cost ratio, MJDL assemblies are widely used in post-frame buildings. Three- or four-layer laminated columns (fig. la) are now used in most post-frame buildings, and the vast majority of roofs are supported with metal plate connected (MPC) trusses. Stacked beams (fig. lb) are finding increased use as rafters, and built-up girders (fig. 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 the 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, fire resistance, thermal conductivity, fatigue resistance, electrical conductivity, space utilization, and level of interference with plumbing, HVAC and electrical hardware.

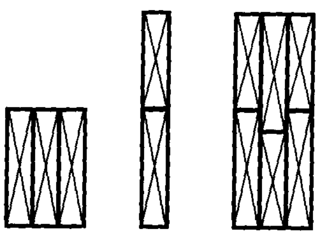

Figure 1-Cross-sections for typical (a) laminated column, (b) stacked beam, and (c) built-up girder.

STACKED BEAMS

Stacked beams are formed by using metal plate connectors to join 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,b; Emanuel et al., 1987), they are frequently used as rafters in large dairy freestall barns. When properly designed and supported, stacked beams can handle considerably larger bending moments than can high grade 38 × 235 mm or 38 × 286 mm (nominal 2 × 10 in. or 2 × 12 in.) members. They are favored over MPC trusses in freestall barns because of their "clean" appearance and because birds can't perch on them.

It is not uncommon for the depth to thickness ratio of MJDL stacked beams to exceed 10. When components are this slender, compression edge support is needed to minimize lateral torsional buckling due to bending loads. In preliminary laboratory tests in which identically sized members were used to form stacked beams (fig. 2a), insufficient lateral bracing resulted in a lateral shifting of

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the compression member relative to that of the tension member (fig. 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 (fig. 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 (i.e., that there is complete composite action). 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 fabrication tolerances and assumptions such as complete composite action (Brakeman, 1998).

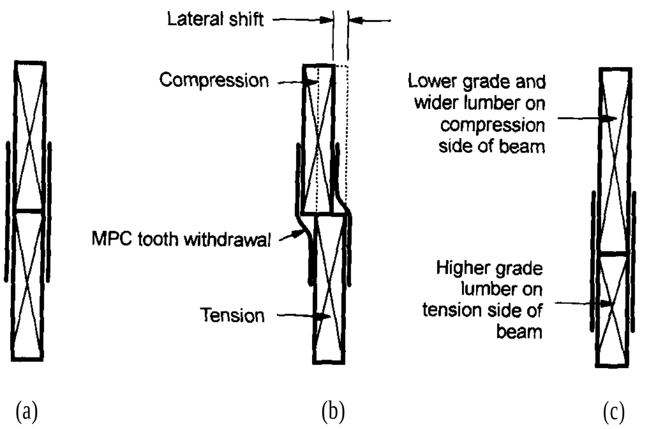

Figure 2-Stacked beams with identically sized members and insufficient lateral bracing: (a) unloaded, and (b) under high bending load. (c) Stacked beam designed to minimize cost and lateral shifting.

BUILT-UP GIRDERS

When two or more stacked beams are laminated together, the resulting assembly is referred to as a built-up girder (fig. 1c). The moment-carrying capacity of a built-up girder is generally at least as great as the sum of the moment-carrying capacities of the individual stacked beams or layers. This is because (1) laminating increases the effective width of the assembly which increases lateral stability under load, (2) load-sharing occurs between individual layers, and (3) 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 5 m).

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, Wisconsin, 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 job site 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 lumber 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, generally it is 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 et al., 1993).

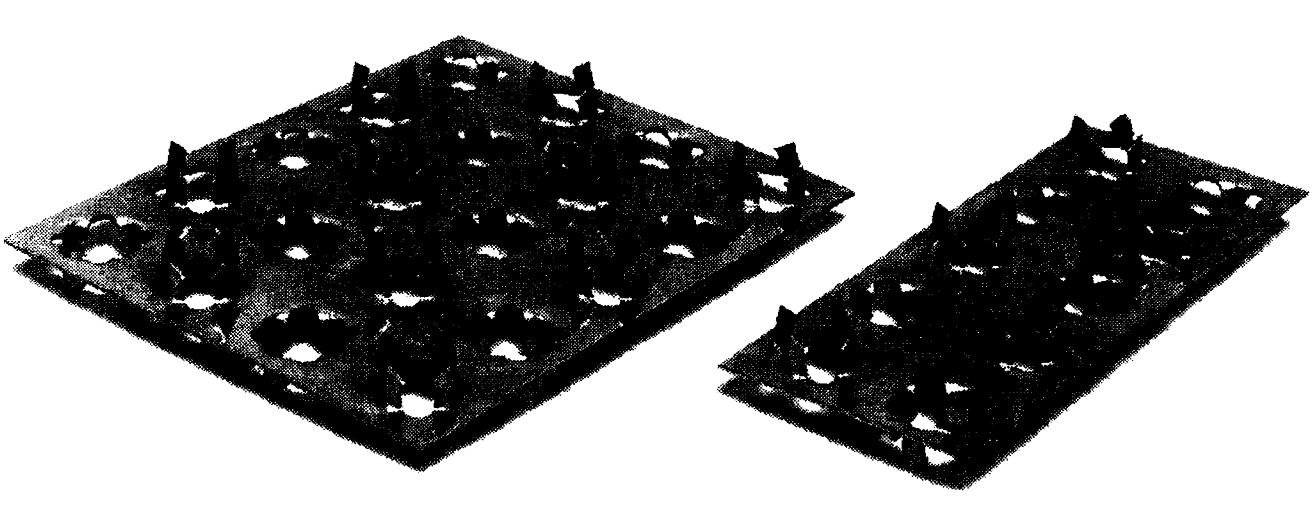

Figure 3-Shear transfer plates (STPs) with plug density of 1 plug/in.^{2}.

Although STPs are principally used to transfer shear between layers, they can be used like MPCs to connect two wood members that have been butted together. Using a 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 that builds up in the flat (non-tooth portion) 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.

STP-Laminated Girder Development

Research conducted in the early 1990s on STP connections (Wolfe et al., 1993) and STP-laminated columns (Bohnhoff et al., 1993) demonstrated the shear transfer efficiency of the Jack Walters & Sons' STP. This research led to the subsequent design of the STP-laminated girder (fig. 5c) as a potential replacement for the nail-laminated design being used by Jack Walters & Sons (fig. 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.

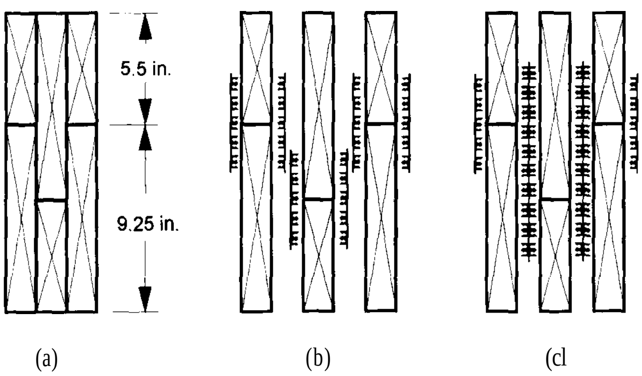

Figure S-Built-up lumber girder showing (a) member lay-up, (b) metal plate connectors (MPCs), and (c) inner MPCs replaced with shear transfer plates (STPs).

Research Objectives and Scope

The objectives of this research were to:

1. Determine the bending strength and stiffness of two STP-laminated girder designs (each with a different arrangement of end-joints), and 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

Girder Design

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 38 × 140 mm and 38 × 235 mm (nominal 2 × 6 in. and 2 × 10 in.) members was selected (fig. 5a). This produced an arrangement with a width of 115 mm (4.5 in.) and a depth of 375 mm (14.75 in.). Overall girder length was fixed at 7.9 m (26 ft).

The second step in girder design was joint pattern selection. The first goal in this process was to select an ideal pattern that would maximize 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 no tension side joints within 2.1 m (7 ft) of midspan when edge A is in compression, and a minimum joint spacing of 0.9 m (3 ft). 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 235-mm (nominal 10-in.) wide member (since its ends are furthest from girder midspan), member 8 was selected to be a 140-mm (nominal 6-in.) wide member. This assignment determined the size of all the remaining 12 members. Lastly, it should be noted that 6.1 m (20 ft) 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 6.1 m stock.

The second goal in joint pattern selection was to select a *less than ideal* pattern. Initially the thought was to create a design with 50 to 100% more end-joints than the pattern in figure 6. However, after 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,

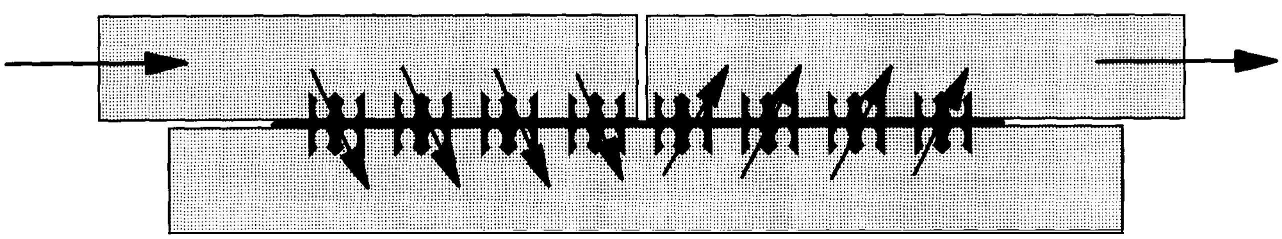

Figure 4-Load transfer around a butt joint via a shear transfer plate.

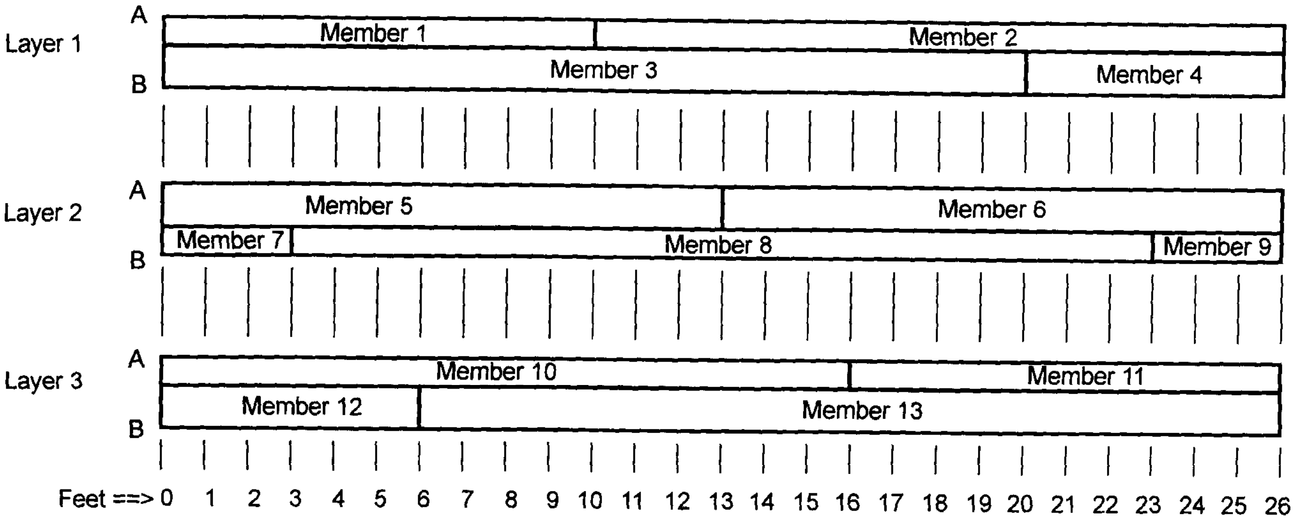

Figure 6–End-joint location for test assemblies.

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 1.2 m (4 ft) 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 an "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 152- × 254-mm (6- × 10-in.) 20-gage MPCs with an on-center spacing of 0.61 m (2 ft). This pattern was duplicated on each side of each layer. In addition, a 356- × 229-mm (14- × 9-in.) 16-gage MPC was embedded into each side of each end-joint. Individual layers were connected with 3.3- × 70-mm (0.131- × 2.75-in.) pneumatically driven nails spaced every 150 mm (6 in.) on each side of the assembly.

Plate locations for the experimental STP-laminated girders are shown in figure 8. By placing 127- × 254-mm (5- × 10-in.) 20-gage STPs vertically with an on-center spacing of 0.30 m (1.0 ft), the amount of steel in shear at each edge joint was approximately the same as that for the nail-laminated girder design (fig. 7).

Throughout the remainder of the article, 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.

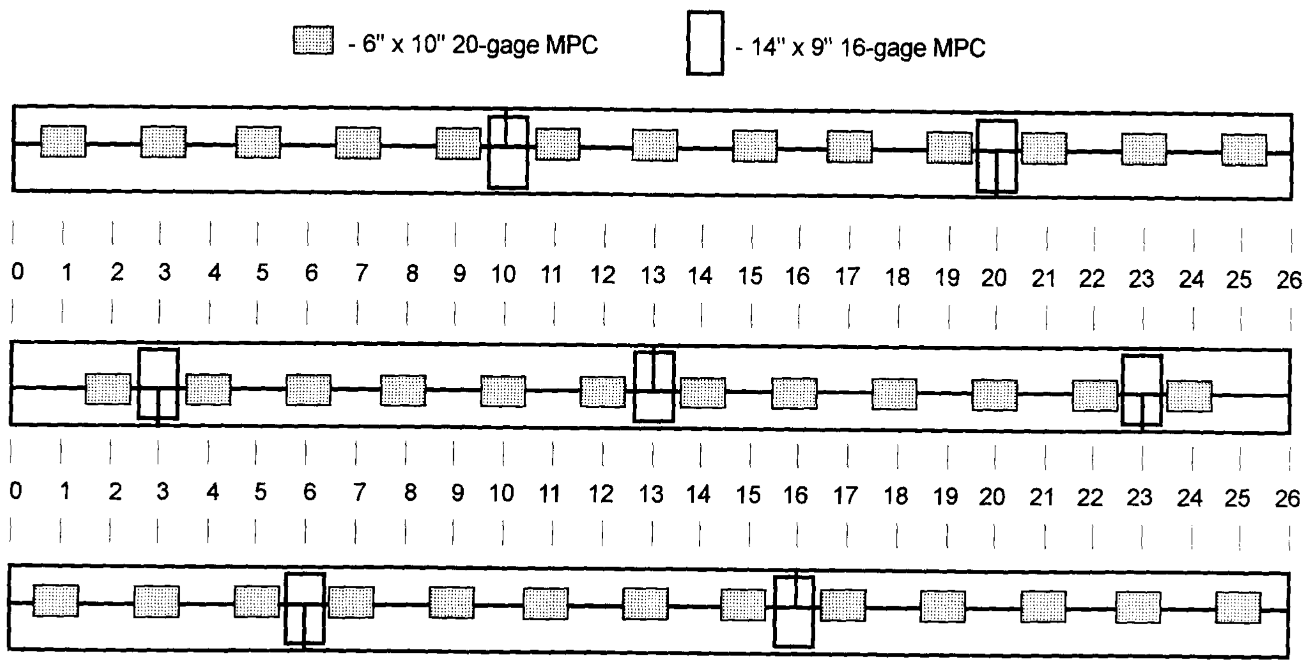

Figure 7–Metal plate connectors location in nail-laminated girders. Same pattern both sides.

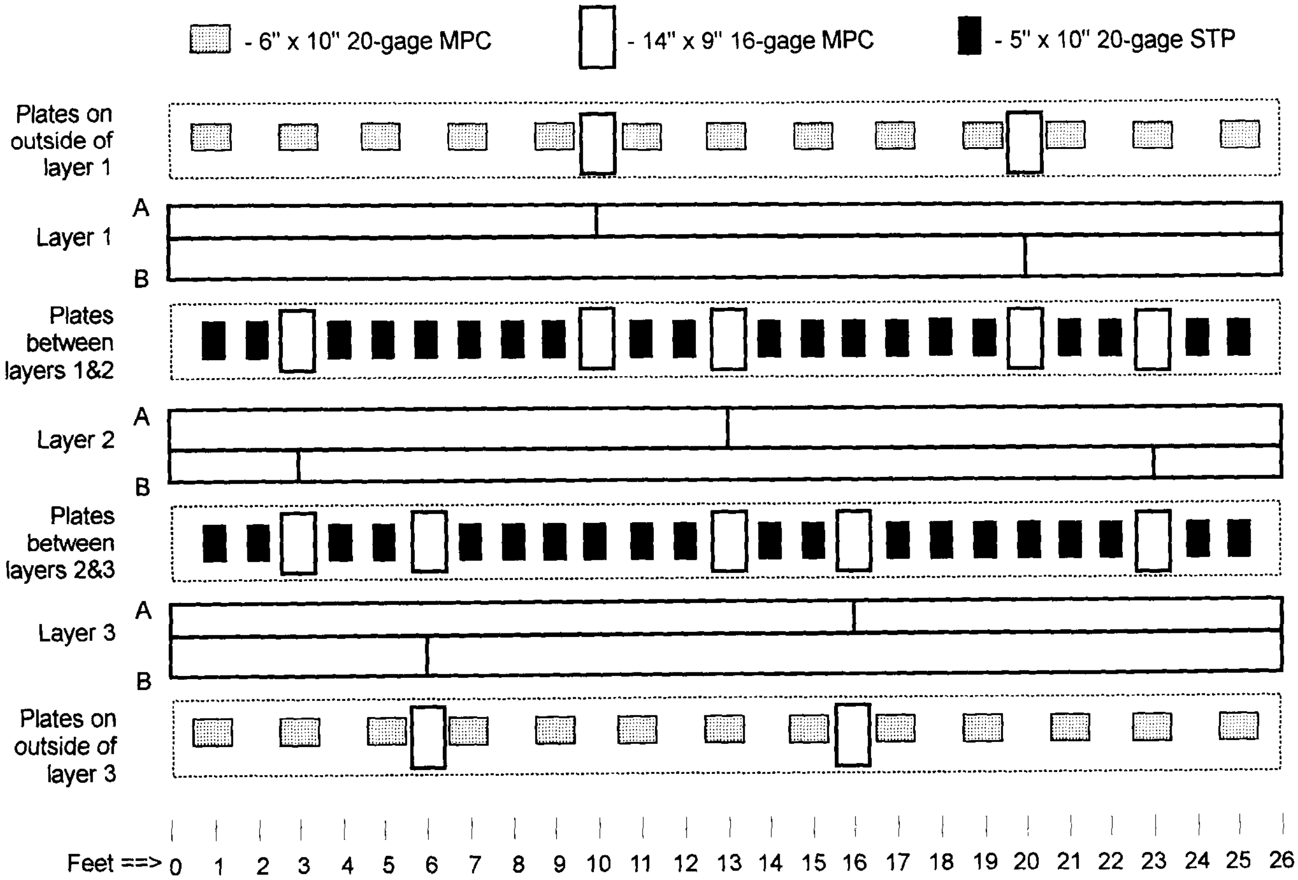

Figure 8–Location of shear transfer plates and metal plate connectors in STP-laminated test assemblies.

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 and thirty-eight pieces of 38 mm × 140 mm × 6.1 m lumber, and an equal number of 38 mm × 235 mm × 6.1 m lumber were obtained for this study. This was enough lumber to build 10 replications of each design. All lumber was machine stress rated, kiln-dried (KD-19) Southern Pine. The 140-mm (nominal 6-in.) wide lumber was grade-stamped 2400f-2.0E. The 235-mm (nominal lo-in.) wide lumber was grade-stamped 2250f-1.9E.

All lumber was stored inside a Jack Walters & Sons' manufacturing facility for approximately one year. Each piece was then given an identification number, measured, weighed, the moisture content taken at three locations with a resistance-type moisture meter, and the modulus of elasticity (MOE) determined using a flatwise, transverse vibration (TV) technique.

To begin the allocation process, 120 members were randomly selected from each group of 138. Next, 60 of the 235-mm-wide pieces were randomly selected and each cut into a 3.96 m, a 1.83 m, and a 0.30 m piece (13, 6, and 1 ft pieces) with the 0.30 m pieces being discarded. Similarly, 60 of the 140-mm-wide pieces were randomly selected and each cut into a 3.96 m, a 1.83 m, and a 0.30 m piece (with the 0.30 m pieces also being discarded). Thirty more 140-mm-wide pieces were randomly selected and each cut into two 3.05 m (10 ft) pieces. This cutting left 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 were associated with 235 mm wide lumber that was 3.96 m (13 ft) long (fig. 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 three 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 powered nailer. No special fixturing or clamping was 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 (fig. 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.

After girder fabrication was completed, it was discovered that three 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 10 mm/min (0.40 in./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 displacements 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 5 cm (2 in.). A computer-based data acquisition system was used to record load-point deflections and load data at 0.5 s intervals. Immediately prior to load application, wood moisture content was checked. These measurements were made at two locations (typically near each load point) on both sides of each assembly.

Results

Lumber Properties

Lumber properties are compiled in table I. This table lists mean values and corresponding coefficients of variation for specific gravity and flatwise, transverse

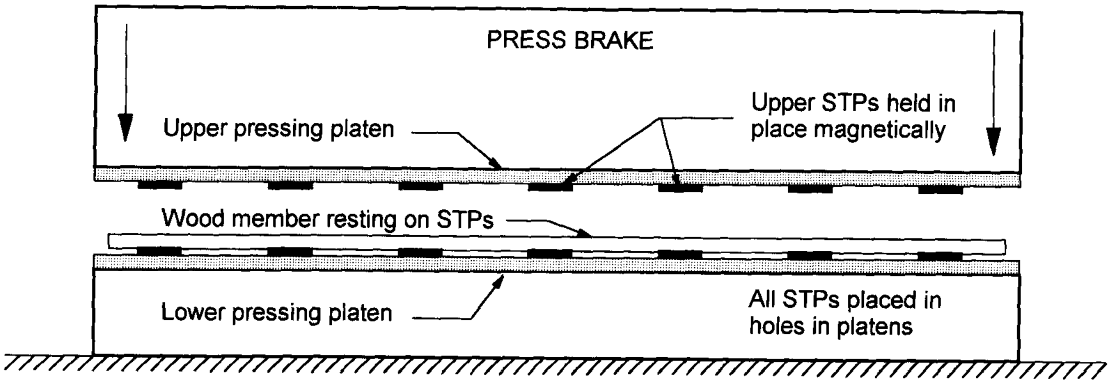

Figure 9–Use of press brake to install plates in center layer of STP-laminated girder.

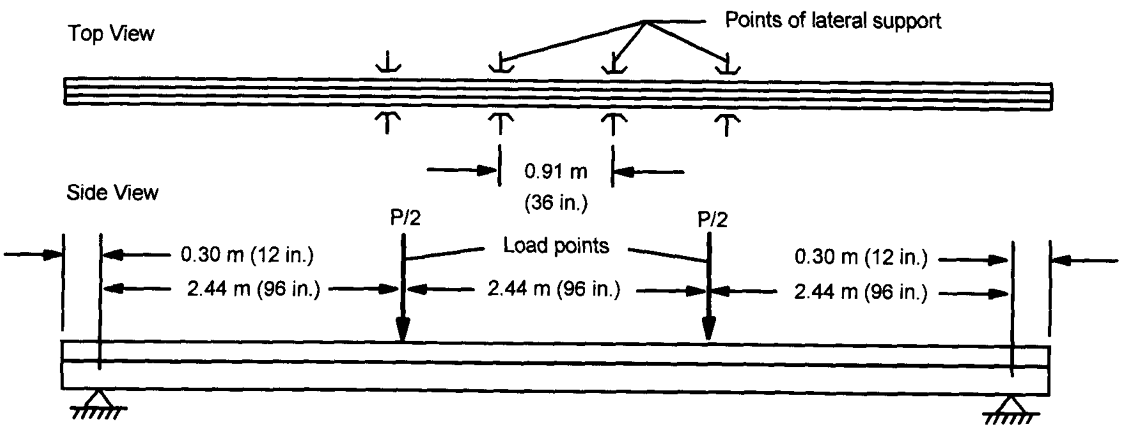

Figure 10–Location of load points, support reactions, and points of lateral support for three-layer built-up lumber girder tests.

vibration MOE. Specific gravity was calculated for each piece of lumber as:

$$SG=W/[V \times (1 + M/100)] \quad (1)$$

where SG is specific gravity based on mass at a 0% moisture content and volume at moisture content M; M is moisture content, percent dry basis, at time of girder fabrication; W is mass at moisture content M; and V is volume at moisture content M (calculated from length, width and thickness measurements).

The moisture content of the lumber at the time of fabrication averaged 13.2%. At test time the average moisture content was 11.1%.

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 4.5 and 31 kN (1000 and 7000 lb). This 26.5 kN (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 1. Lumber properties

Lumber Size (mm × mm)	No. of Pieces	Modulus of Elasticity* Mean, GPa	($\times 10^6$ lb/in.^{2})	COV, %	Specific Gravity† Mean	COV, %
38 × 140	120	15.2	(2.21)	12.9	0.53	8.3
38 × 235	120	16.1	(2.34)	10.5	0.57	7.2

* For lumber at an average moisture content of 13.2%. MOE determined by flatwise, transverse vibration (TV).
† Specific gravity based on bone-dry mass and volume at time of fabrication. Calculated using equation 1.

Table 2. Girder initial stiffness and ultimate midspan bending moment

Repl. No.	Initial Stiffness (kN/m)* NAIL-A	STP-A	STP-B	Ultimate Midspan Bending Moment (kN-m)† NAIL-A	STP-A	STP-B
1	1150	1100	1070	123	139	105
2	1130	‡	1170	125	‡	107
3	1120	‡	1120	121	‡	116
4	1100	‡	1080	132	‡	102
5	1140	1190	1200	134	122	107
6	1160	1190	1260	133	150	102
7	1100	1050	1090	124	136	102
8	1180	1110	1100	139	146	101
9	1150	1130	1170	118	114	100
10	1105	1200	1165	112	130	105
Mean	1133	1139	1144	126	134	105
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 4.5 and 31 kN. Multiply by 5.71 to convert to lb/in.
† Multiply by 737.6 to convert to ft-lb.
‡ 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. Two common failures in addition to wood failures included: MPC failure at tension side joints, and shear of MPCs along the edge between the 140-mm- and 235-mm-wide 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 2×10* means that two different 38 × 235 mm (nominal 2 × 10 in.) 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 perpendicular-to-grain. Also, no attempt was made to distinguish between initial and secondary failures because of the number of simultaneously appearing failures, and the inability to identify when failures occurred in the middle layer.

Discussion

Lumber Modulus of Elasticity

Flatwise, transverse vibration MOE values are often used to predict static edgewise MOE and other lumber properties, For this reason, the transverse vibration MOE values were compared to MOE values from the NDS (AF&PA, 1997), and found to exceed the NDS values by 10.5% and 23% for the 38 × 140 mm and the 38 × 235 mm lumber, respectively. This difference can be explained in part by the fact that MSR lumber is sorted to maintain minimum strength and MOE thresholds. For wider width lumber, strength typically governs and as a result, average measured MOE is often higher than the nominal NDS design value.

Bending Stiffness

The mean initial bending stiffness of girder designs NAIL-A, STP-A and STP-B were calculated to be 1133, 1139, and 1144 kN/m, 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

Table 3. Failure location and frequency*

Failure Description	Replicate Number 1	2	3	4	5	6	7	8	9	10	Average Frequency	Specimens Exhibiting Failure (%)
Design NAIL-A												
MPC failure – tension side joint	2	1	1	1	-	-	1	-	1	1	0.8	70
MPC shear between stacked members	-	1	-	-	-	-	-	-	-	-	0.1	10
Wood failure – tension side 2×10	1	1	2	1	1	2	-	1	2	3	1.4	90
Wood failure – compression side 2×10	-	1	-	-	1	-	-	1	-	-	0.3	30
Wood failure – tension side 2×6	-	-	-	-	-	-	1	1	-	1	0.3	30
Wood failure – compression side 2×6	1	-	-	-	1	-	1	-	-	-	0.3	30
Design STP-A												
MPC failure – tension side joint	-	†	†	†	2	1	1	-	2	-	0.9	57
MPC shear between stacked members	-	†	†	†	-	-	-	-	-	-	0	0
Wood failure – tension side 2×10	2	†	†	†	1	1	1	1	1	2	1.3	100
Wood failure – compression side 2×10	-	†	†	†	-	-	1	-	1	-	0.3	28
Wood failure – tension side 2×6	-	†	†	†	-	1	-	-	1	-	0.3	28
Wood failure – compression side 2×6	-	†	†	†	-	-	1	1	-	-	0.3	28
Design STP-B												
MPC failure – tension side joint	3	2	3	3	3	3	1	1	2	2	2.3	100
MPC shear between stacked members	-	-	1	2	-	-	-	-	1	-	0.4	30
Wood failure – tension side 2×10	-	1	-	1	-	-	1	-	1	-	0.4	40
Wood failure – compression side 2×10	-	-	-	-	2	2	-	1	-	1	0.6	40
Wood failure – tension side 2×6	-	-	-	-	1	-	1	1	1	1	0.5	50
Wood failure – compression side 2×6	-	-	1	-	-	-	-	-	-	-	0.1	10

* Numbers in table indicate number of members or number of joints exhibiting same failure.
† Girder incorrectly fabricated and omitted from analysis.

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 the stiffness of designs STP-A and STP-B to differ when both are under low loads.

Averaging the three mean initial stiffness values yields a stiffness value of 1139 kN/m. If the girders were assumed to be homogenous solids 114 mm (4.75 in.) thick and 375 mm (14.75 in.) deep, this stiffness value would be associated with an apparent edgewise bending MOE of 13.7 GPa (1.99×10^6 lb/in.^{2}). This is about 12.5% less than the average MOE of the lumber as determined by flatwise, transverse vibration. This difference can be attributed to lack of complete composite action in built-up girders, and to fact that MOE values determined by flatwise vibration may be over-estimating 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 126, 134, and 105 kN-m, 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 that 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 have similar bending stiffnesses, and are forced by a load distributing element to displace the same amount. When interlayer 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 2×10. 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 2.4 m) of each girder. The reversal of load from edge A to edge B (fig. 6) resulted in these end-joints being subjected to tensile forces instead of compressive forces. In five (50%) of the STP-B assemblies, MPC failure occurred at all three of these tension side end-joints. In three (30%) of the 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 for the traditional allowable stress design format are generally calculated by dividing the fifth percentile point estimate of ultimate

Table 4. Fifth percentile point estimates of ultimate midspan bending moment

	Bending Moment (kN-m)		
Distribution	Design NAIL-A	Design STP-A	Design STP-B
Normal	112.6	112.8	96.7
Lognormal	113.1	113.4	97.2
2-Parameter Weibull	110.2	111.8	91.9
Average	112.0	112.6	95.2

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 point 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 53.3, 53.6, and 45.3 kN-m for designs NAIL-A, STP-A, and STP-B, respectively.

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 homogenous solid of equivalent size and shape. For the girders tested in this study, an equivalent homogenous solid would have a section modulus of 2674 cm^3 (163.2 $in.^3$). Dividing this value into the previous allowable design bending moments yields allowable effective design bending stresses of 19.9, 20.1, and 17.0 MPa (2890, 2910, and 2460 $lb/in.^2$) for designs NAIL-A, STP-A and STP-B, respectively. All three of these values exceed the NDS design values of 16.5 and 15.5 MPa (2400 and 2250 $lb/in.^2$) associated with the 38 × 140 mm and 38 × 235 mm (nominal 2 × 6 and 2 × 10 in.) lumber, respectively. However, when the 16.5 and 15.5 values are increased 15% (to 19.0 and 17.8 MPa) for repetitive member use, they both exceed the 17.0 MPa value associated with Design STP-B. As an aside, it should be noted that past research has shown the NDS repetitive member factor of 15% to be low for unspliced mechanically laminated dimension lumber.

Finally, the effective design bending stresses calculated for the built-up girders should be applied with caution. 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 may not be representative of their respective grades.

SUMMARY

Twenty-seven built-up girders were tested to failure in bending. Each girder was 7.9 m (26 ft) in length and contained three layers, with each of these layers comprised of stacked 38 × 140 and 38 × 235 mm (nominal 2 × 6 and 2 × 10 in.) members. Nails were used to join individual layers in 10 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 homogenous solid with an edgewise bending MOE equal to the average lumber MOE as measured by transverse vibration during girder 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 (AF&PA allowable stress design) for assemblies with critical end-joints located in compression regions was 20.0 MPa (2900 $lb/in.^2$). This suggests that high strengths can be obtained by proper design of built-up girders.

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