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Current and Future Applications of Mechanical Fasteners for Light-Frame Wood Structures

Proceedings of Mechanical Fasteners Plenary Session at the Forest Products Research Society Annual Meeting

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Compiled by Leslie H. Groom

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CONTENTS

Introduction to Mechanical Fasteners Plenary Session
Leslie H. Groom 1
Wood Connection Design: 1991 National Design Specifications and Beyond
T. E. McLain and D. G. Pollock, Jr. 3
Eccentricities in Metal-Plate Connected Joints
Michael H. Triche 5
Determining Failure Mechanism in Bolted Joints Using Acoustic Emissions
Marcia Patton-Mallory 7
Strength and Stiffness of Shear Transfer Plate Connections
Ron Wolfe and Dave Bohnhoff 9
Mechanical Connections: The Weak Link in Light-Frame Wood Construction
Gary J. Klein 13

Introduction to Mechanical Fasteners Plenary Session

Leslie H. Groom

The title of this publication—"Current and Future Applications of Mechanical Fasteners for Light-Frame Wood Structures"—is the theme of this plenary session from the 1991 annual meeting of the Forest Products Research Society (FPRS). This theme was chosen to address current issues in the forest products industry: changing design codes, fastener performance, and legal responsibilities. It is hoped that this session helped provide a forum for forest products industries influenced by mechanical fasteners and also for research agencies, resulting in an increased exchange of current knowledge and concerns for the future.

Mechanical fastener research has blossomed in recent years as a result of the changing wood base and of liability issues and foreign competition. Liability has become a significant problem because of the increasing frequency and severity of litigation, resulting in the financial collapse of some businesses and increased insurance premiums for all. A key defense against litigation is proper assessment of product performance. This information is needed not only by manufacturers of mechanical fasteners but also by other potential lawsuit defendants, such as design agencies and trade associations. Economical and innovative designs of engineered wood structures (whose properties are largely governed by mechanical fasteners) must be coupled with product performance to be competitive in world markets.

The presentations given at this plenary session were chosen because of their timeliness and importance. Individual articles in this publication are based on the presentations. The first article, by Tom McLain, addresses the current revamping of the National Design Specifications (NDS) for mechanical fasteners, which are undergoing a fundamental change. The new design specifications will be based on the European concept of yield theory and will be incorporated in the 1991 NDS manual. Tom discusses the NDS changes and how they will affect light-frame construction practices.

Product performance is addressed in the next three articles, with topics progressing from theoretical modeling to mechanical behavior of mechanically fastened joints. The modeling article, by Mike Triche, focuses on the analysis and design of truss-plate joints. Marcia Patton-Mallory explores experimental techniques for determining the integrity of mechanical fastened joints in her article. She discusses the use of acoustic emissions to determine the failure modes of bolted joints. In the product performance article, Ron Wolfe addresses the mechanical behavior of shear transfer plates. Traditionally, this mechanical fastener has been limited to laminated beam columns; however, shear transfer plates have exceptional mechanical properties and may prove useful in other engineered systems such as timber bridge decks. Ron details the load capacity of stress transfer plates and the factors that affect their performance.

The final article of this publication is by Gary Klein, who discusses the consequences of a lack of product performance understanding by examining the collapse of structures as a result of failure of the mechanically fastened joints. Gary presents forensic case histories and discusses the liability associated with the failures.

The contributors selected were chosen on the basis of their knowledge of the subject area and respect among their colleagues. They represent industry, academia, and U.S. Government research facilities. I should note that there had not been an FPRS plenary session on mechanical fasteners for several years, and the enthusiasm of the contributors was very high. A high level of interest was shown by both industry and researchers. I hope the session was enjoyable and educational and that discussions continue.

Leslie H. Groom is a research forest products technologist at Alexandria Forestry Center, U. S. Department of Agriculture, Forest Service, Southern Forest Experiment Station, Pineville, LA 71360.

Wood Connection Design: 1991 National Design Specifications and Beyond

T. E. McLain, D. G. Pollock, Jr.

Design provisions for engineered wood connections are changing substantially for the first time in over 50 years. These changes can be found in the upcoming 1991 edition of the National Design Specification (NDS) for Wood Construction and in a proposed load and resistance factor or design (LRFD) Specification for Engineered Wood Structures. The purpose of this presentation is to outline some of these changes.

BEHAVIORAL EQUATIONS

A major change in design basis is the adoption of a well-verified theoretical base for lateral yield strength criteria for connections using nails, wood screws, bolts, and lag screws. Yield strength is defined as shown in figure 1. The European Yield Models (EYM) are used to calculate design strength based on the bending resistance of the fastener, the crushing strength of wood or member material, joint geometry, and a set of assumed mechanical relationships. The EYM describe a set of possible yield modes for a single fastener under lateral load (see fig. 2) for double-shear connections. Results of EYM for common connections are calibrated to 1986 NDS criteria to minimize change. However, because of the wholesale change in design equations and calibration to only one point in

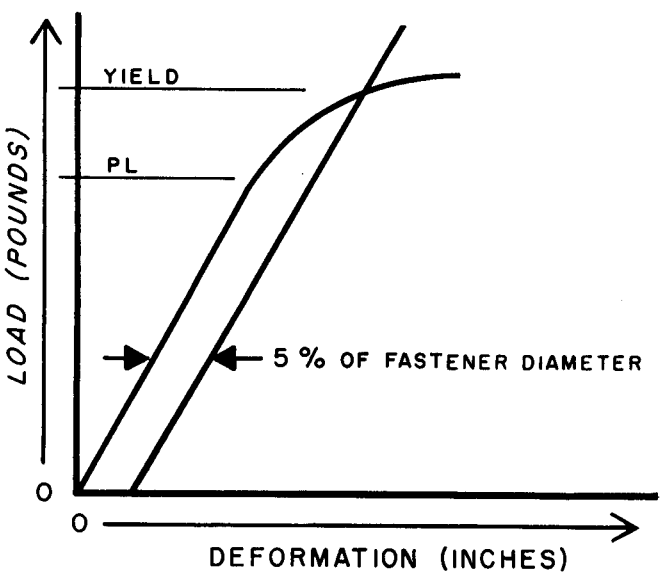

Figure 1.—*Fastener yield strength as based on 5 percent of the nominal shank diameter. Note: PL = Proportional Limit.*

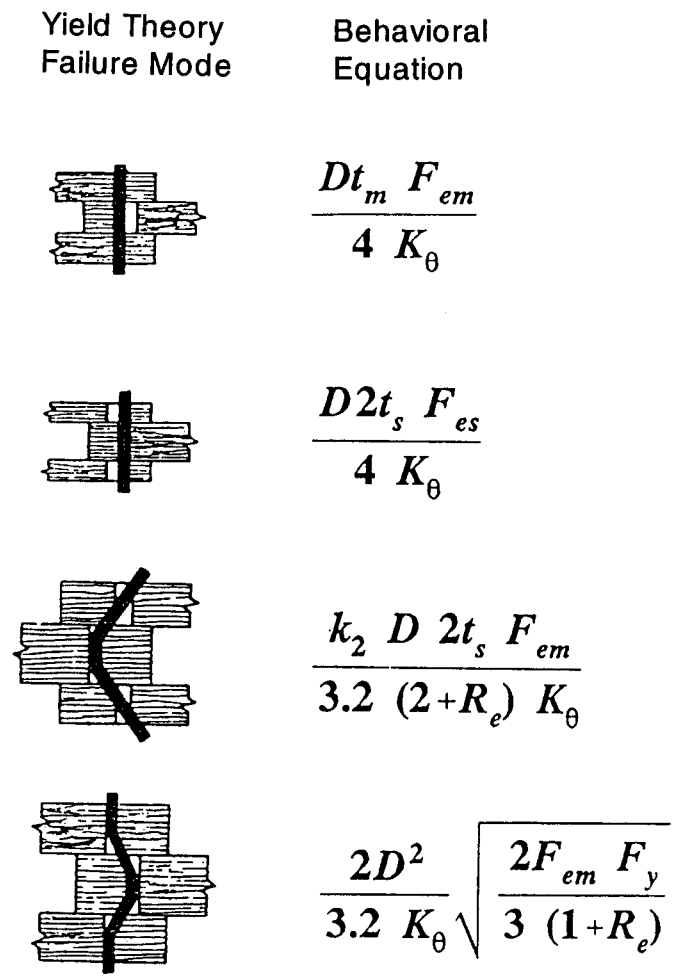

Figure 2.—*Yield modes and corresponding behavioral equations for double-shear connections with dowel-type fasteners.*

T. E. McLain is professor, Virginia Polytechnic Institute and State University, Blacksburg, VA 24061; D. G. Pollock, Jr. is director of engineering, National Forest Products Association, Washington, DC 20036.

design space, there will be changes in future connection design. For lateral loading of connections, the EYM require a dowel-bearing strength, F_e, and fastener yield F_y. The F_e is directly related to specific gravity, which is used as an index variable for species effects and replaces the traditional species connection groups.

Specific Gravity

Eight species were tested in the National In-Grade Test Program, and the average specific gravities *(SG)* obtained have been adopted. For all other species, the methods of ASTM D2555 with a 10-percent grouping limit are proposed. These methods result in slight changes from historic practice. The principal change is not with the level of *SG* chosen for a species but elimination of species groupings in favor of a more continuous strength-*SG* relationship.

Dowel-Bearing Strength, F_e

Dowel-Bearing Strength, F_e, is determined through a simple compression test of a dowel into a predrilled half-hole. For solid wood, the USDA Forest Service Products Laboratory (FPL) has obtained the relationship shown in table 1. For connections loaded at an angle to the grain, F_e is found with Hankinson's formula. This is more convenient than applying Hankinson's formula when solving the connection yield strength, and it gives essentially the same results.

Fastener Yield Strength, F_y

The yield strength of the fastener, F_y, on a 5-percent nominal shank diameter (D) offset basis may be found by standard tests or from published values. Historically, with allowable stress design, 45 ksi (310 MPa) has been assumed as the F_y of common steel bolts. This is not contraindicated by recent research from FPL. Research at Virginia Polytechnic Institute and elsewhere indicates that for common wire nails, F_y = 130 ksi minus 214D where D = nominal diameter. For a 16-penny nail with D = 0.162 inch, F_y = 95 ksi.

MODIFICATION FACTORS

Most of the historic modification factors used to design strength for end-use conditions remain unchanged. Exceptions in the NDS include minor adjustments to the penetration factor. The only substantive change in the NDS is with the group action or multiple fastener factor, C_g, which is now in an equation format and also tabulated. This factor is changed from MDS-86 to more accurately reflect the supporting research results, which show that C_g is a function of connection stiffness. Consequently, C_g for bolted connections and for split ring/shear plate connections is now different.

CALIBRATION

Historically, engineered wood connections have been safe. The NDS-91 design values were derived from yield equations and calibrated to the safety levels found in NDS-86. Calibration was done by a yield mode that minimized change but did not eliminate it. The yield equations were directly calibrated to steel side plate connections with a low-to-intermediate range of bolt L/D ratios. The NDS-91 nail connection design values were generally increased over the NDS-86 values.

Some connection design strengths increased, and some decreased as a result of the combined effect of all changes. Overall, the design strength of most connections with bolts and lag screws is within plus or minus 15 percent of the NDS-86 levels. Little or no change is seen for split ring/shear plate connections or in the design strength of connections relying on shank withdrawal.

Table 1.—*Dowel-bearing strength equations for various fastener types and grain angles*

Fastener type	Grain angle	Dowel-bearing strength
Nails, spikes, wood screws	All grain angles	$F_e = 16600\ SG^{1.84}$*
Bolts, lag screws, large dowel	Parallel-to-grain	$F_e = 11200\ SG$
Bolts, lag screws, large dowel	Perpendicular-to-grain	$F_e = 6100\ SG^{1.45}D^{-0.5}$

*SG = specific gravity based on ovendry weight and volume; D = nominal shank diameter (inches); and F_e = dowel-bearing strength (psi)

Eccentricities in Metal-Plate Connected Joints

Michael H. Triche

Design methods for metal-plate connected joints have evolved rapidly over the past several years. Specific requirements are published by the Truss Plate Institute, Design Specifications for Metal Plate Connected Wood Trusses, TPI-85, *(1)*. A new specification, TPI-92, *(2)*, will be published in the near future. Although TPI-92 includes a number of advances in connector plate designs and testing requirements, it does not cover specific methodology for handling eccentricities in connections. Typically, joints are designed such that eccentricities are very small, and the resulting moment in the connection is negligible compared with the direct shear. However, in certain odd situations, connections may have to be designed to resist moment.

The method presented here accounts for eccentricities in connections. The proposed method determines forces on individual teeth of the metal plate using the nonlinear load-displacement relationships of the teeth, which are determined by standard test *(2)*. Variations in stiffness because of varying angles of grain and tooth orientation are considered.

Several linear models have been used in the past to design plates for moment resistance. The most common is the polar moment of inertia approach, which was used successfully for years in steel construction. This method, however, is very conservative when compared to actual experimental behavior; the American Institute of Steel Construction currently publishes tables for standard connections used in steel buildings *(3)* that are based on a nonlinear "instantaneous center of rotation" method, not the linear polar moment of inertia approach. The nonlinear method predicts joint capacity with much greater accuracy, and the connections designed by this method are considerably more economical. The key to the method is the use of the nonlinear load-displacement characteristics of the fasteners, which allows for the ultimate strength of the connection to be determined and a factor of safety to be applied to the ultimate strength of the entire connection rather than to individual fasteners.

When a load is applied to metal-plate joints, the load is transferred from the wood into the steel plate by the lateral resistance of the teeth of the plate. For a joint under axial load, each tooth of the plate is assumed to share the load equally. This assumption, while not completely accurate, is consistent with the design procedure used for nailed joints. Under pure moment, however, the teeth revolve around a common point, and the teeth farthest from the center of rotation undergo much greater movement than those near the center.

The use of a single linear load-displacement relationship for the teeth, which does not account for wood grain direction or tooth orientation, results in tooth forces directly proportional to the distance of the tooth from the center of rotation. This is the basis of the polar moment of inertia approach, which has often been proposed. However, the results obtained by this method are inaccurate. In reality, the lateral resistance of the teeth of a plate is distinctively nonlinear and direction dependent. This nonlinearity is beneficial to joints under eccentric load because the force for a given displacement is lower than that calculated if a linear load-displacement relationship were assumed.

Connections must also be designed on the basis of ultimate strength rather than allowable stresses. Under the allowable stress approach, the design capacity of the connection is based on the load at which the first fastener of the connection reaches its allowable load. In this procedure, a factor of safety is taken on the individual fasteners and not on the capacity of the entire joint. At these low load levels, the fasteners are still in the linear range, and the benefit of the nonlinear load-displacement behavior is not realized. This approach is unnecessarily conservative. The preferred method is to determine the ultimate strength of the connection and to apply the factor of safety to the entire connection.

To illustrate the differences in the polar moment of inertia approach and the method proposed here, five moment splices were considered. The specimens consisted of a pair of 2 by 4's spliced together with metal

Michael H. Triche is assistant professor, Civil Engineering Department, University of Alabama, Tuscaloosa, AL 35487.

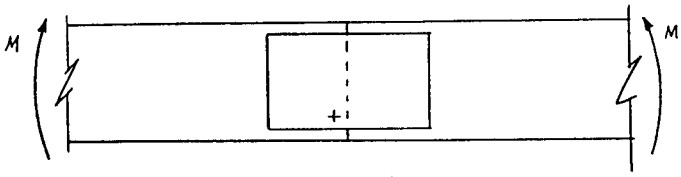

Figure 1.—*Diagram of a moment splice joint; the "+" shows the location taken for the center of rotation used in the analysis of the left connection area.*

plates as shown in figure 1. Each connection was analyzed by the two methods; plate size, orientation, and the resulting moment capacities are shown in table 1. For both methods, the center of rotation was taken to be at the location of the bottom corner due to the wood-to-wood bearing that occurred at the bottom edge of the 2 by 4 ends.

The joints analyzed were previously tested in another study *(4)*; the results are shown in table 2. Also given in table 2 are the ultimate capacities as predicted by the proposed method, along with percent errors. Although the proposed method of analysis underestimated the actual capacity of the joint in all but one case, it was in general agreement with the actual capacity. Referring back to table 1, note the linear, polar moment of inertia method predicts considerably less moment capacity and therefore would result in extremely conservative designs.

In conclusion, methodology is proposed for analyzing metal-plate connections subjected to moment. The method is based on ultimate strength and requires load-displacement information, which is available from standard test procedures. The method predicts actual joint behavior much more accurately than other commonly used linear methods. Also, this method can be used to aid in the development of reliability-based design procedures for wood trusses.

Table 1.—*Comparison of proposed method with linear method for moment capacity of metal-plate connected joints with varying plate size and orientation*

Plate size	Plate orientation	Proposed method	Linear method
Inches	*Degrees*	*In-lb*	*In-lb*
2.5 by 5.25	0	6,091	4,871
3.5 by 3.0	90	5,511	4,796
3.0 by 3.5	0	5,254	4,395
5.0 by 3.0	90	8,211	7,110
3.0 by 5.25	0	8,533	6,902

Table 2.—*Comparison of experimental and proposed model results for moment capacity of metal-plate connected joints with varying plate size and orientation*

Plate size	Plate orientation	Experimental capacity	Predicted capacity	Percent error
Inches	*Degrees*	*In-lb*	*In-lb*	
2.5 by 5.25	0	7,689	6,091	–20
3.5 by 3.0	90	6,435	5,511	–14
3.0 by 3.5	0	6,798	5,254	–22
5.0 by 3.0	90	7,920	8,211	+4
3.0 by 5.25	0	9.454	8,533	–10

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Determining Failure Mechanism in Bolted Joints Using Acoustic Emissions

Marcia Patton-Mallory

Understanding the mechanisms by which failure occurs in wood connections can lead to better criteria for specifying their design load and design details. The research described considered failure mechanisms that occur in wood connections containing steel bolts. Failure processes were evaluated during laboratory testing of connections. The sounds generated during the failure process—termed acoustic emissions (AE)—were recorded and compared with observed differences in failure modes.

Connections with the shorter end distance failed in a brittle splitting mode with only minor crushing of the wood beneath the bolt. This was the primary failure mode for connections that had a small enough bearing length that the bolt did not yield during the test. The pattern of AE during these tests was a sudden increase in event rate, high amplitude events, and high energy content of events just prior to failure fig. 1.

Matched connections with longer end distances failed by wood crushing beneath the bolt. In contrast to the short end distance failure mode AE pattern, emissions were primarily of a lower amplitude, at a fairly constant event rate, and with an increasing event rate at the yield point (fig. 2). Higher amplitude events occurred only after considerable deformation in the joint and only when small shear cracks formed adjacent to the crushed wood.

The connection yield point was determined graphically as the intersection of a line parallel to the major linear portion of the load-deformation curve but offset by 5 percent of the bolt diameter. The yield point determined in this matter corresponded closely to the increase in AE events when bolted connections failed in a yielding (crushing) failure mode.

This study showed that AE patterns correlated closely with observed brittle and ductile failure modes in bolted connections. However, care should be taken to minimize small knots and drying cracks because these defects will cause unwanted AE during the test.

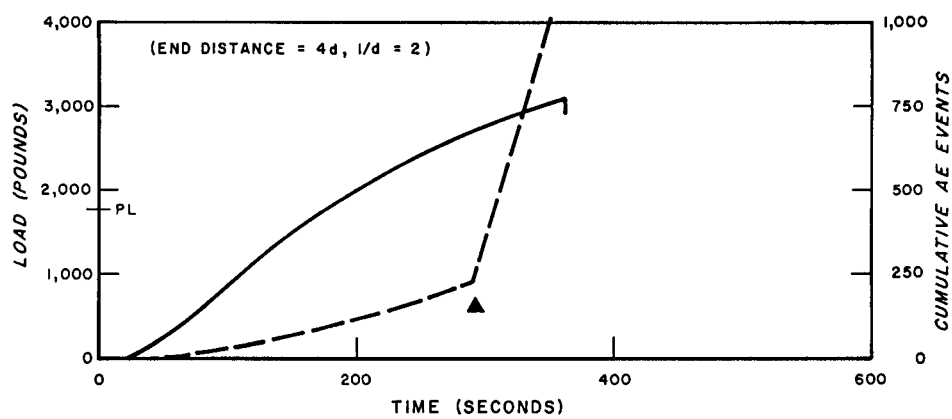

Figure 1.—*Brittle failure mode: acoustic emission increase below yield in bolted connection with a short end distance. Note:* d = *bolt diameter;* l = *bolt bearing length;* PL = *proportional limit; and* ▲ = *acoustic emission event rate change.*

Marcia Patton-Mallory is assistant director, U.S. Department of Agriculture, Forest Service, Rocky Mountain Forest and Range Experiment Station, Fort Collins, CO 80526.

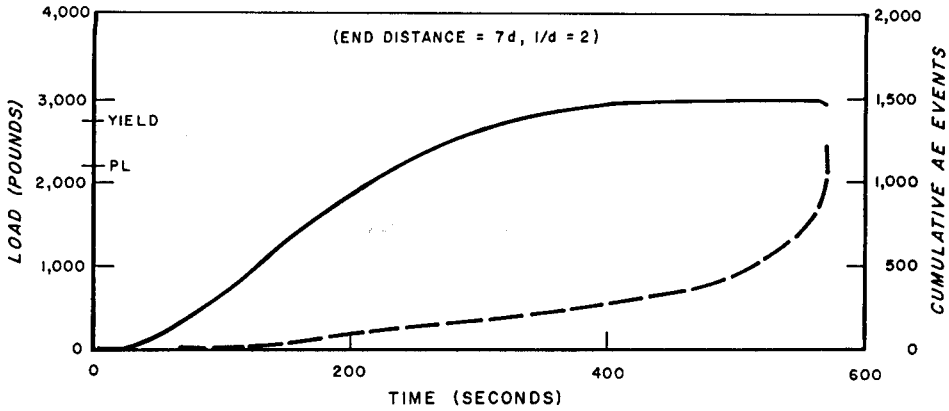

Figure 2.—*Ductile failure mode: acoustic emission increase near yield in bolted connection. Note:* d = *bolt diameter;* l = *bolt bearing length;* PL = *proportional limit; and* ▲ = *acoustic emission event rate change.*

Strength and Stiffness of Shear Transfer Plate Connections

Ron Wolfe and Dave Bohnhoff

A common problem faced by post-frame builders is the difficulty of getting stress-rated solid sawn 8- by 8-inch timber posts. One solution to this problem is to use posts fabricated by laminating stress-graded 2- by 8-inch material. Glue-laminating may be prohibitively expensive, but mechanically laminated posts can be fabricated at a reasonable cost. Mechanically laminated columns have the additional advantage that they allow the builder to limit the use of treated material to the embedded portion of the post.

One approach to mechanically laminating wood posts is to use a double-sided connector plate called a shear transfer plate (STP) (fig. 1). Sandwiching STPs between adjacent layers provides needed post stiffness by limiting interlaminar shear displacement and limits weakening effects of butt joints by transferring shear forces around them.

Although the concept of a double-sided nail plate for shear transfer between layers in a laminated assembly is not new, its use has been limited to custom applications. As a result, there have been few published reports dealing with STP design properties or the structural capacity of STP connected assemblies. To model the behavior of mechanically laminated assemblies, data are needed to characterize the load-slip behavior of the connections.

The objective of this study was to add to the limited available data base on strength and stiffness of STP connections. Variables considered include fabrication methods, plate size and type, joint confinement, load orientation with respect to the grain, and wood species/specific gravity.

EXPERIMENTAL METHODS

The study was conducted in three phases, summarized in figure 2. The first phase concentrated on the effects of plate size and fabrication method. The second phase dealt with the effects of joint boundary conditions. The third phase focused on effects of plate type, wood species, and load orientation with respect to the wood grain.

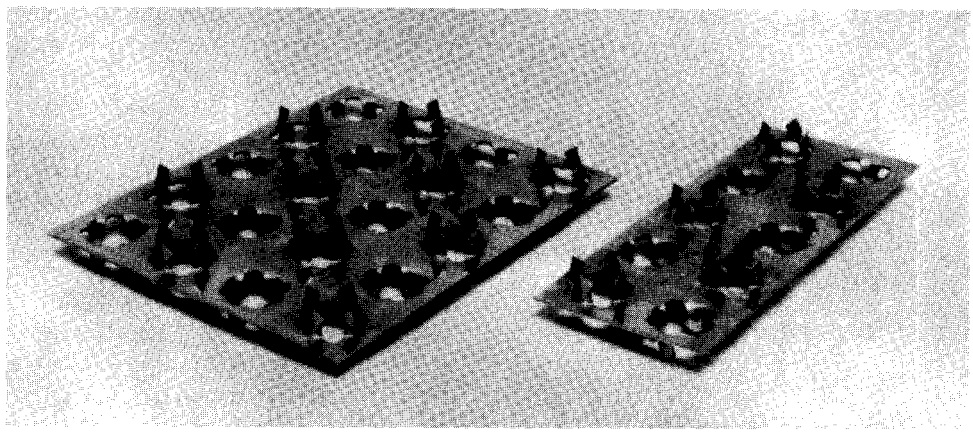

Figure 1.—*Two sizes of shear transfer plates were used in the initial phase of the study. One was a 4- by 5-inch plate containing 10 plugs per side, and the other was a 2- by 5-inch plate containing 5 plugs per side.*

Ron Wolfe is a research engineer, U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI 53706; Dave Bohnhoff is a professor, Department of Agricultural Engineering, University of Wisconsin, Madison, WI 53706.

Phase	Species	Plate		Pressing	Test[a]	Load to grain	Joint code
		Size	Type				
I	SYP	2 by 5	GALV	ONE STAGE	1	⊥	25-1
		4 by 5					45-1
		2 by 5		TWO STAGE			25-2
		4 by 5					45-2
II	SYP	4 by 5	GALV	TWO STAGE	1		WOA
					2		AHO
					3		ALO
					4		AHI
III	SYP	4 by 5	SS	TWO STAGE	1	⊥	SYP-S-E
			GALV				SYP-G-E
			SS			⊥⊥	SYP-S-A
			GALV				SYP-G-A
	SPF		SS			⊥	SPF-S-E
			GALV				SPF-G-E
			SS			⊥⊥	SPF-S-A
			GALV			⊥⊥	SPF-G-A
	DF		SS			⊥	DF-S-E
			GALV			|	DF-G-E
			SS			⊥⊥	DF-S-A
			GALV			||	DF-G-A

[a] Confinement conditions:
[1] No test apparatus used.
[2] Apparatus used with no restraint.
[3] Apparatus used with 20 psi restraining pressure.
[4] Apparatus used with 40 psi restraining pressure.

Figure 2.—*Experimental design: Species considered included southern yellow pine (SYP), spruce-pine-fir (SPF), and Douglas-fir (DF); plates include stainless steel (SS) and galvanized steel (GALV) 20-gauge shear transfer plates with one four-tooth rosette (plug) per square inch. Twenty-two joints were tested for each configuration.* Joint code: S=stainless steel, G=galvanized steel, E=perpendicular, A=parallel.

The control joints referenced in each phase were fabricated using the two-stage pressing method, southern pine lumber, and 20-gauge, 4- by 5-inch galvanized metal STPs. Each joint was composed of three parallel laminations, connected using two STPs. Load, applied perpendicular to the grain of the middle member, was transferred through the STPs to the outer members, which were supported on the bed of the test machine. In two-stage pressing, plates were completely pressed into both sides of the center member, using special steel pressing plates that fit over the STP. The outside pieces of wood were then positioned over the STP and pressed into place. In one-stage pressing, STPs were positioned between the wood members, and the four wood-plate interfaces were pressed simultaneously.

SUMMARY OF TEST RESULTS

The control joint had an average maximum load capacity of 455 pounds per plug with a coefficient of variation of only 8 percent. For design purposes, the average load at 0.015 inches slip was 390 pounds per plug with a coefficient of variation of 9 percent. Test

results suggest that these values are significantly affected by fabrication method, wood specific gravity, plate size, and joint boundary conditions. No significant effect was noted for load-to-grain orientation or for galvanized versus stainless steel STPs.

Joints fabricated using single-stage pressing methods required greater total fabrication energy and had less stiffness and strength than those fabricated using the two-stage process. For obtaining joints of comparable appearance, fabrication pressure requirements for single-stage pressing were on the order of 25 percent greater for single-stage pressing than for two-stage pressing. The single-stage 4- by 5-inch plate joints were only 75 percent as stiff and 85 percent as strong as the two-stage joints. For the 2- by 5-inch plates, this effect on joint stiffness was slightly less (89 percent), but strength effect was the same.

	Joint code	Stiffness		Strength	
		lb/in	Rel	lb/plug	Rel
Control	II-WOA	50790	1.05	449	0.99
	III-SYP-G-E	50641	1.05	470	1.08
	I-46-2	42628	.90	447	.96
	Total	46241	1.00	465	1.00
Species	III DF-G-E	41022	.55	448	.98
	III SPF-G-E	28582	.59	359	.79
Pressing	I 46-1	35975	.75	381	.84
Orientation	III SYP-G-A	34708	.72	472	1.04
Plate Type	III SYP-G-E	31449	.55	511	1.12
Plate Size	I 26-2	29776	.62	558	1.28
Confinement	II ANO	25122	.52	266	.58
	II ALO	25364	.53	440	.97
	II AMI	24601	.51	484	1.06
Species/Plate Type	III DF-6-E	82667	.58	452	.99
	III SPF-8-E	18536	.59	360	.79
Species/Orientation	III DF-G-A	32241	.57	300	1.10
	III SPF-G-A	13352	.58	380	.88
Orientation/Plate	III SYP-G-A	25965	.54	488	1.07
Species-Orient-Plate	III DF-G-A	25537	.53	501	1.10
	III SPF-G-A	11467	.24	337	.74
Pressing and Plate Size	I 26-1	26682	.55	402	1.08

Figure 3.—*Relative values of average joint load capacity. The stiffness values given were derived as the stiffness (K) parameter derived on a per-plug basis to fit the Foschi nail-plate model. Strength values represent the maximum load for the joint uniformly distributed to the shear transfer plate plugs.*

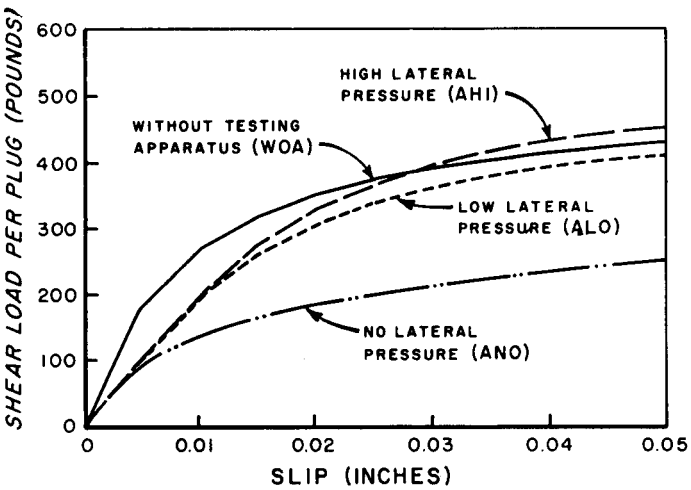

Figure 4.—*Confinement effect on joint performance. Joints tested in the confinement apparatus with no lateral pressure had no external forces acting to resist lateral movement.*

Results of tests conducted to evaluate plate size effect are inconclusive. The 2- by 5-inch plate size appeared to give a per-plug load capacity 25 percent greater than that of the 4- by 5-inch plate. This difference was significant at the 90-percent confidence level. Differing modes of failure, however, suggest that this difference in strength may have been partially influenced by the size of the joint wood members relative to the size of the STPs used. For the 4- by 5-inch plate joints, wood deformation caused by bearing stresses influenced joint strength.

Joint strength and stiffness are directly related to specific gravity. Relative values of average joint load capacity (fig. 3) measured at incremental displacements were fairly constant over the range from 0.005 to 0.05 inch and roughly equal to average relative specific gravity values. Differences between the southern pine and Douglas-fir joint strengths were not significant at the 90-percent confidence level, but the southern pine joints were significantly stronger than the spruce-pine-fir joints.

Test boundary conditions have a major effect on failure mode and joint strength (fig.4). Joints tested with no lateral restraint had a failure load 55 percent less than those tested with some form of restraint. Joint strength was not improved significantly by substituting lateral pressure for bearing friction or by increasing lateral pressure beyond 20 psi.

CONCLUSIONS

Our research on strength and stiffness of STP connections led to the following conclusions:

- The two-stage method of joint fabrication is preferable to single-stage pressing.
- Further study is needed to verify/quantify a plate size effect. Additional studies should consider wood member size relative to STP size as a variable.
- Results affirm a strong and direct relationship between joint strength and member specific gravity.
- The effect of load orientation with respect to wood grain direction appears to be species dependent. Although it was not significant at the 90-percent confidence level, the effect of load orientation was most noticeable for Douglas-fir joints.
- Further study is needed to relate test boundary conditions to in-use conditions.

Mechanical Connections: The Weak Link in Light-Frame Wood Construction

Gary J. Klein

Metal plate-connected trusses are one of the most widely used structural systems in residential and light commercial construction. When properly designed and built, plate-connected trusses can be expected to perform very well under most conditions.

In spite of advances in quality control, poorly installed connector plates often result in trusses that are much weaker than intended. Among the most common problems are mislocation and partial embedment of connector plates. These and other problems with truss connections are described in this paper. Methods for repairing deficient joints are also presented.

CONNECTOR PLATE MISLOCATION

Connector plates are sometimes mislocated during fabrication, resulting in a significant reduction in the capacity of the connection. Often the contact area of the connector plate on the diagonal web member is less than required to develop the computed diagonal force at service loads (fig. 1).

Sometimes individual plates are mislocated. This problem is particularly likely where very small plates are used. On the other hand, plate mislocation can be systematic. Generally, this condition is not apparent until the shear stresses between the connector plates and wood members are computed.

Related deficiencies include connector plates that are missing, improperly oriented (fig. 2), or undersized. Also, wane or knots in the joint region can significantly diminish the capacity of the connection (figs. 3,4). These problems can result in joint failures and sometimes collapse, although it has been my experience that wood truss systems have a remarkable ability to hang together in spite of numerous connection failures.

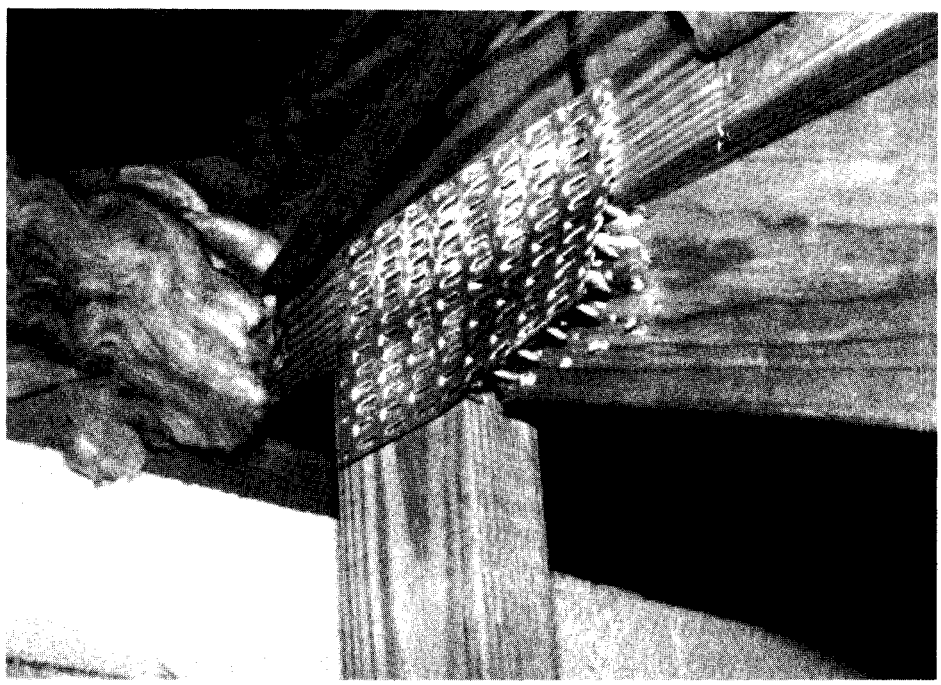

Figure 1.—*Pull-out of overstressed connector plate.*

Gary J. Klein is a consultant at Wiss, Janney, Elstner Associates, Inc., Northbrook, IL 60062.

Figure 2.—*Pull-out of misoriented plate.*

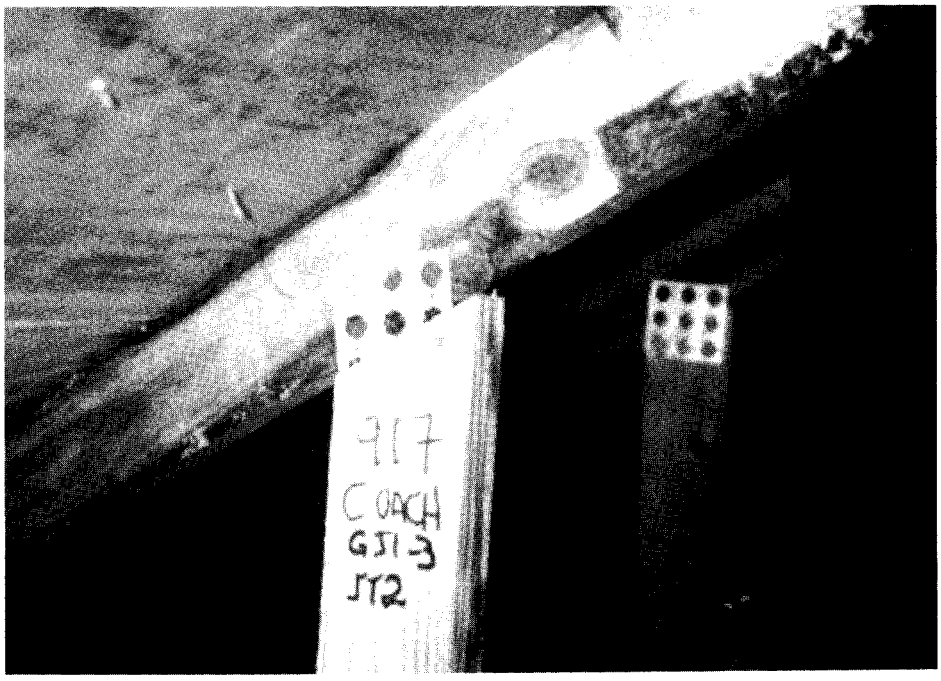

Figure 3.—*Wane in joint region.*

Figure 4.—*Diagonal failure at knott below connector plate. Note that plate does not cover the entire joint, concentrating the diagonal force at the knot.*

PARTIAL PLATE EMBEDMENT

Often connector plates are not properly pressed when trusses are fabricated, or the plates tend to "walk out" as wood dries during service (fig. 5). Partial embedment can also occur because of a mismatch in member thickness or width (fig. 6). The latter problem is particularly common on 2- by 4-inch parallel chord trusses.

Partial embedment is characterized by a uniform gap between the connector plate and the wood member. Joints with partially embedded plates are much weaker than joints with fully embedded plates. The gap causes weak axis bending of the teeth. The problem is particularly severe for 20-gauge plates loaded along the length of the plate. A limited research program has shown that a 1/16-inch gap below a 20-gauge plate reduced the joint strength by roughly 50 percent (fig. 7).

REPAIRS

Joint deficiencies can be repaired with plywood gussets attached to one or both sides of the truss using thru-bolts or lag bolts. A limited testing program was conducted to verify the effectiveness of 3/4-inch plywood gussets (APA B-B Plyform, class 1) connected to southern pine 2 by 4's using 1/2-inch bolts.

Recommended design values are as follows:

Connection Type	Suggested design value
3/4-inch plywood each side (double shear) and 1/2-inch diameter thru-bolts	900 lb per bolt
3/4-inch plywood one side (single shear) and 1/2-inch diameter lag bolts	350 lb per bolt

These values are based on a proportional limit load and allow for time effects and a safety factor. A reduction per the National Design Specification (NDS) for multiple fasteners in a row should be considered, although the test results did not show an apparent strength reduction for up to three thru-bolts and six lag bolts in a row.

A hydraulic C-clamp can also be used for a variety of repairs. The most frequently used C-clamp repair is to repress partially embedded connector plates (fig. 8). The C-clamp can also be used to install new connector plates at truss joints following shoring (if required) and removal of the existing plates.

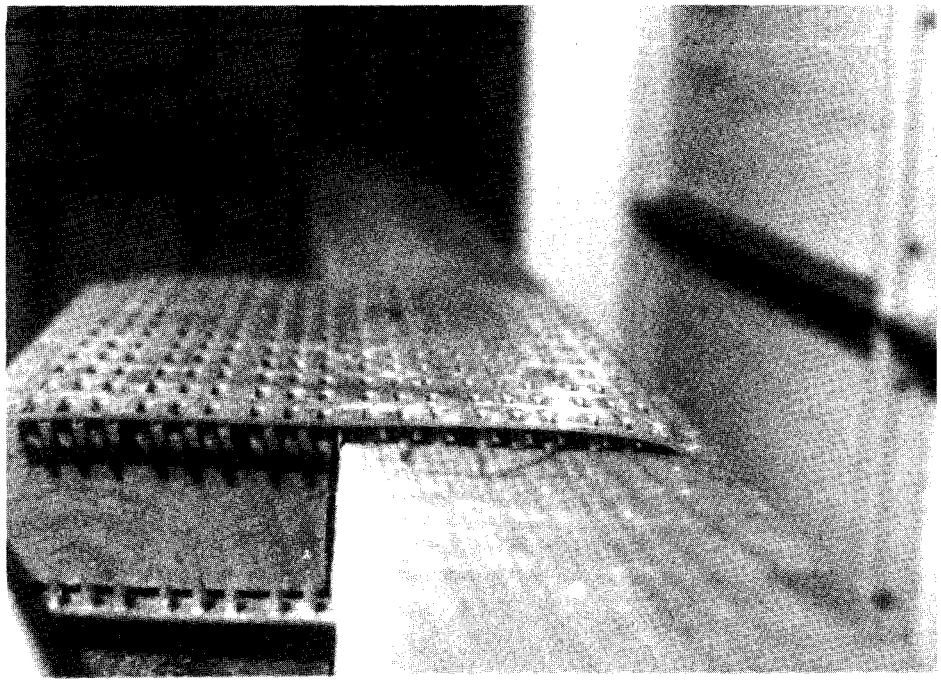

Figure 5.—*Partially embedded connector plate.*

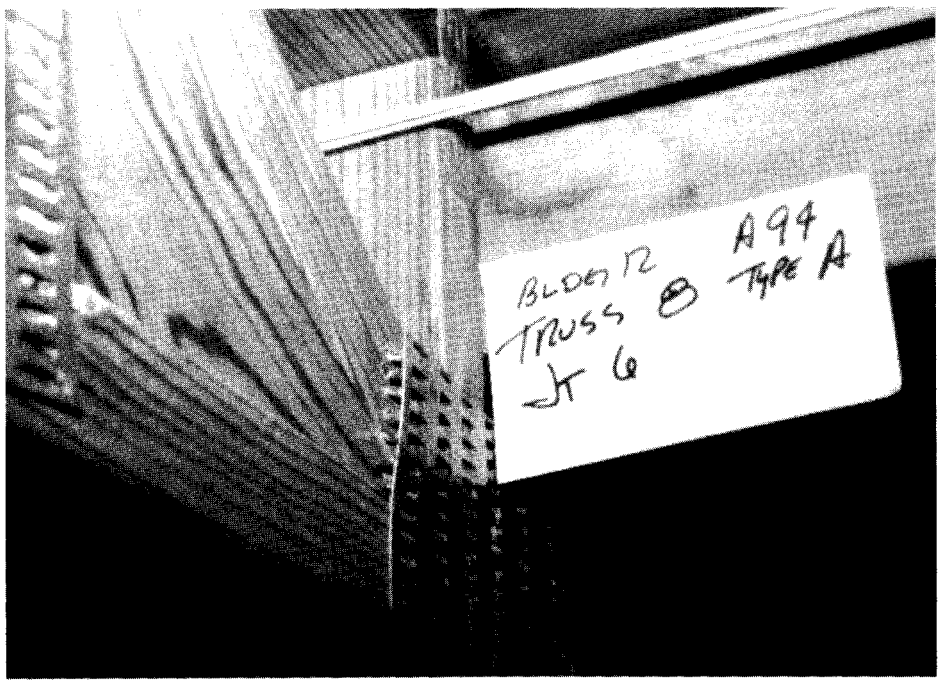

Figure 6.—*Pull-out of partially embedded connector plate due to mismatch of 2 × 4 width.*

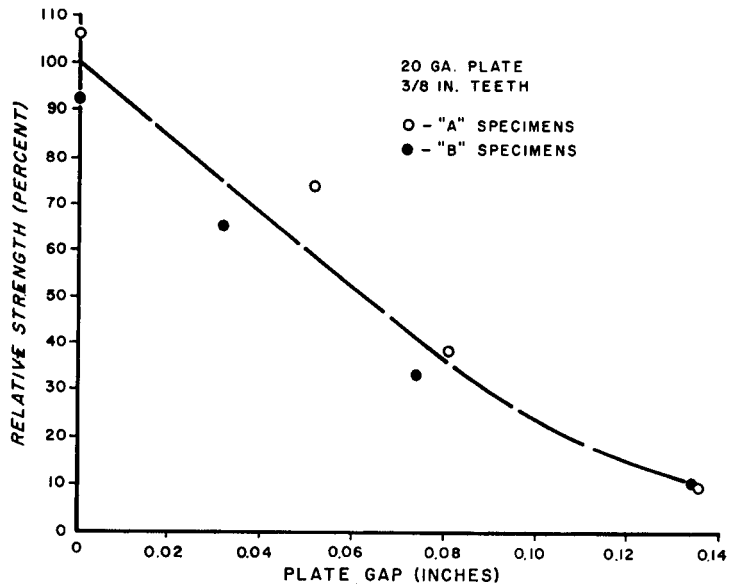

Figure 7.—*Relative joint strength versus plate gap for a 20-gauge plate with 3/8-inch teeth, where ○ and ● represent specimen data from separate sample sets.*

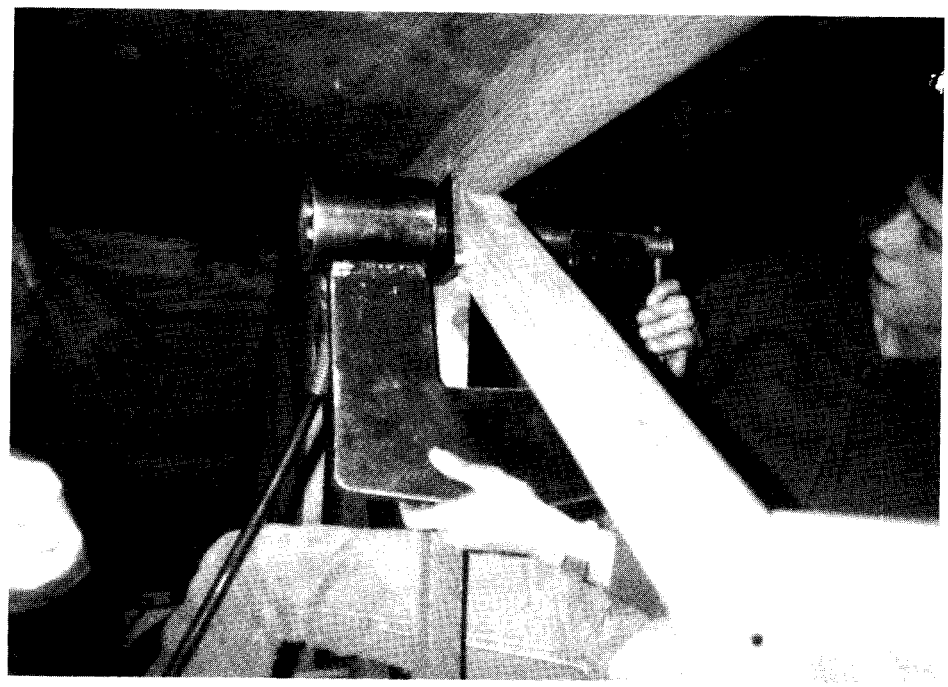

Figure 8.—*Field repressing of partially embedded connector plates.*

CLOSING REMARKS

The kinds of problems described are becoming less frequent. In recent years, the plate-connected truss industry has made very significant advances in establishing and enforcing quality. It is hoped that these advances continue and that ongoing research and development will strengthen the reliability of mechanical connections in light-frame wood structures.

Groom, Leslie H. 1993. Current and future applications of mechanical fasteners for light-frame wood structures; Proceedings of mechanical fasteners plenary session at the Forest Products Research Society annual meeting. Gen. Tech. Rep. SO-92. New Orleans, LA. U.S. Department of Agriculture, Forest Service, Southern Forest Experiment Station. 18 p.

Articles based on the five presentations at the mechanical fasteners plenary session of the 1991 Forest Products Research Society annual meeting are given. The information presented is aimed at designers of light-frame wood structures and researchers of mechanical properties of fasteners.

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