NAIL LOADS FOR TRUSS CONNECTIONS USING STEEL VS. FIVE-PLY AND FOUR-PLY FIR AND SPRUCE PLYWOOD GUSSETS

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Contribution no. I-730, Received 18 July 1985, accepted 13 March 1986

Turnbull, J. E. and **L. P. Lefkovitch** 1986. Nail loads for truss connections using steel vs. five-ply and four-ply fir and spruce plywood gussets. Can. Agric. Eng. **28**: 167–173.

Traditional five-ply, 12.5-mm Douglas fir plywood has recently disappeared from the Canadian retail lumber market. Now the product is four-ply, 12.2-mm, which has shown consistently poorer performance when tested as connecting gussets for nailed wood roof trusses. This experiment examined some of the implications of the changes in CSA-0121 Douglas fir plywood, and seeks a better alternative gusset/nail combination. The best connection tested was a two-member, three-gusset system using 0.91-mm (20-gage) steel side gussets and a 1.22-mm (18-gage) center gusset giving a unit lateral resistance of 3.15 kN/nail, compared with five-ply Douglas fir plywood at 0.99 kN/nail. Five-ply Douglas fir plywood in turn gave significantly higher ultimate unit lateral resistance (N_u) than four-ply Douglas fir and the spruce plywoods tested. However, at 1.27 mm slip, the unit lateral slip resistance values (N_s) for the Douglas fir plywoods did not show a significant strength advantage over the values for corresponding layups of spruce plywood.

INTRODUCTION

Canada Plan Service (CPS) roof trusses have traditionally been designed with nominal 12.5-mm Douglas fir (D. fir) plywood gussets on two sides of 38-mm wood frames, nailed from both sides with 4 × 64-mm concrete nails, based on Turnbull and Theakston (1964). As an alternative to factory-prefabricated press-plate trusses, this has given strong, predictable connections suitable for on-site construction by relatively unskilled farm and construction workers.

Turnbull and Theakston (1964) also tested spiral "Truss Gusset" nails made especially for this application by the Steel Co. of Canada. These nails were 4.9 × 65 mm, a little stronger and somewhat cheaper than the 4 × 64-mm concrete nails. However, in some parts of Canada it has been hard to find the Truss Gusset nails; therefore, the CPS trusses were later revised to use the widely available concrete nails.

Recently, Turnbull et al. (1981) gave some alternatives to the simple three-member, double-shear nailed truss joint; they used doubled frame members and three gussets to give three or four shear planes per nail. The center gusset (between frames) was 0.91 mm (20 gage) galvanized steel and the outside gussets were either 0.91 mm steel or 12.5 mm plywood. The nails were 102 mm common spiral nails. The main benefit from using these two-member, three-gusset systems was a doubling of the truss spacing (typically 1.2 to 2.4 m) and a halving of the total number of trusses and nails required to make a given roof. Massé (1985) evaluated axial performance, as well as combined axial-rotational effects, with the same two-member, three-gusset nail system. His main objective was to refine the overall design of CPS doubled roof truss frames by considering all the primary and secondary member stresses as affected by both axial and rotational stiffness characteristics of the joints.

A change in the Canadian Standards Association (CSA) standard, Douglas fir plywood (Anonymous 1978), allows a reduction from five to four veneer plies in the nominal 12.5-mm layup, with a corresponding increase in the veneer thicknesses used. Considering typical lower chord truss gussets stressed primarily in tension parallel to the face grains, this reduces the minimum net thickness of the plies effective in tension from 7.2 mm (five-ply) to only 4.8 mm (four-ply).

Another result of this change to four-ply plywood is that the two center veneers have grains running parallel to each other. The result is effectively a single center veneer, more than twice the thickness of any veneer in the previous five-ply layup. This may, under wet service, expand with much greater force, placing higher preload shear stresses on the glued boundaries. Farm building roof trusses, although normally not designed for wet service, may in fact be subjected to occasional wetting due to winter condensation, and the wet periods can easily coincide with maximum roof snow loads.

Since it is impractical to specify 12.5-mm five-ply in a retail market now supplying only the four-ply product, the current designs for CPS nailed trusses are now in jeopardy. This experiment was undertaken to study some implications of this change of plywood standards and to seek an improved connection system.

The initial short-term strength of truss joints is a consideration, but the long-term performance of the joints is more important. The typical environment in a livestock building may be quite conducive to corrosion of metal fasteners and rotting of truss members within the joints. These long-term performance considerations are beyond the scope of this experiment.

PROCEDURES

Objectives and Test Specimens

It was decided that this experiment would follow the objectives of previous work (Turnbull et al. 1981); that is, to seek an optimum truss connection made with nails and gusset materials easily purchased in Canadian farming communities. To maximize the shear planes penetrated by each nail, the nail length would correspond closely to the total joint thickness. Nails would be driven from both sides of the joints, and they would be spaced to minimize wood splitting.

Figure 1 gives details of the A-frame test specimens and the joints. Each A-frame test simulated two heel joints of a farm roof truss. Tests were similar to those made previously (Turnbull et al. 1981) except that the number of heel joint nails was increased (from 8 to 12 in the lower chords and from 9 to 13 in the upper chords). This made the gussets also liable to failure, on the chance that it might provide some verification of published plywood design stresses as applied to nailed truss joints.

Turnbull et al. (1983) observed that, in a two-member three-gusset nailed joint, the center steel gusset is usually the first to reach failure stress. This is because each

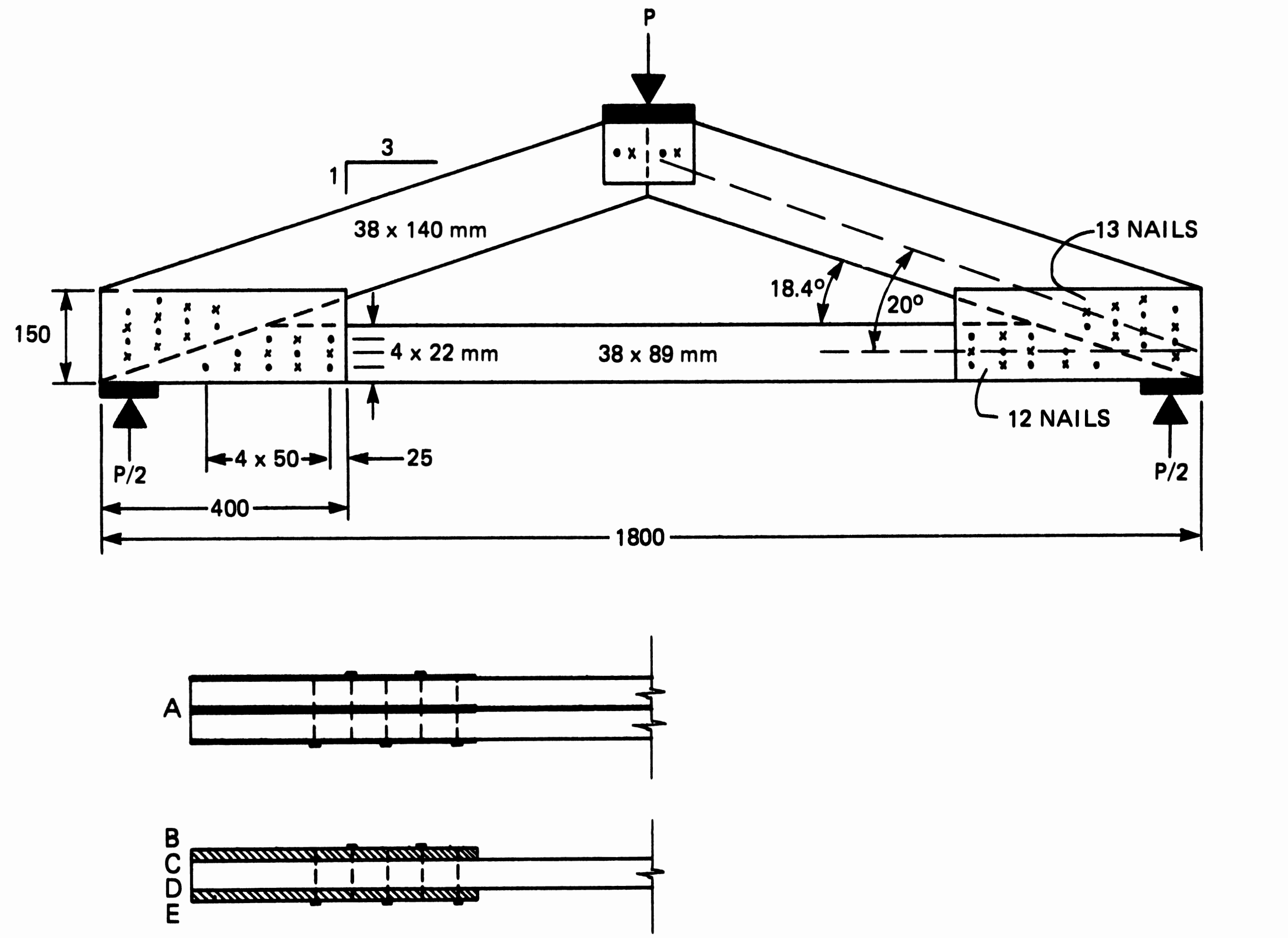

Figure 1. Details of test specimens. (A) 4.4 × 76-mm concrete nails; 0.91-mm galvanized steel side gussets; 1.22-mm galvanized steel center gusset. (B) 4.0 × 64-mm concrete nails; five-ply D. fir gussets. (C) 4.0 × 64-mm concrete nails; four-ply D. fir gussets. (D) 4.0 × 64-mm concrete nails; five-ply spruce gussets. (E) 4.0 × 64-mm concrete nails; four-ply spruce gussets.

nail develops two shear planes about the steel center gusset but only one shear plane at each plywood side gusset. Massé (1985) in fact confirmed with strain gages that about 55% of the total joint force passed through the center steel gusset and only 22.5% passed through each plywood side gusset. In an all-steel gusset system (Fig. 1A, for example), where the nails do not penetrate the third gusset, perhaps up to 2/3 of the force goes to the center gusset.

For this reason system A (Fig. 1) was given a center gusset thicker than the outside gussets, that is 1.22 mm center versus 0.91 mm outside. The previous tests (Turnbull et al. 1981) used 0.91-mm gussets exclusively. In this test the carpenters used a 32-oz club hammer to handle the tougher requirements of penetrating the 1.22-mm center gussets.

If successful in all respects, the all-steel gusset connection A as described in Fig. 1 might replace the double-member truss joints in some current CPS truss designs where plywood side and steel center plates are used (Turnbull et al. 1981, 1983).

Test Procedure

Test frames as detailed in Fig. 1 were assembled using green wood (21.6–27.2% moisture), subsequently dried for about 6 wk in a controlled atmosphere, then load-tested at 15.8–17.7% moisture content. The wood stock was no. 2 or better, S-P-F (spruce-pine-fir) species group, not tested or preselected for density. All three parts for each A-frame were cut from the same 38 × 235-mm plank.

Each of the five tests was replicated six times making 30 specimens. The test apparatus was as used previously (Turnbull et al. 1981) with the hydraulic pump and cylinder calibrated to give 1.91 mm/min average piston velocity. This corresponds to a frame-to-gusset deformation rate of 1.91 × 0.1763 = 0.336 mm/min, chosen to correspond to the testing rate used originally by Turnbull and Theakston (1964). The factor 0.1763 was calculated from the geometry of the A-frame test specimens, using a Williot-Mohr diagram and the assumed elastic moduli of the frames and gussets to convert from piston travel to joint deformation. Thus for the single-member frames with plywood gussets,

$$X = 0.1763\,(D - 0.1525P) \qquad (1)$$

and for the double-member frames with steel gussets,

$$X = 0.1763\,(D - 0.076P) \qquad (2)$$

where X = joint deformation (mm), D = piston travel (mm), P = piston load (kN).

Joint deformation X in Eqs. 1 and 2 is a calculated average single-connection displacement. It is based on the assumption that the upper chord-to gusset part and the gusset-to-lower chord part displace equally, acting like two similar connections in series. In fact, the nails per connection and the forces at each connection are not identical, but the forces per nail are nearly identical. The terms (D − 0.1525 P) and (D − 0.076P) adjust for the slight increase in piston travel attributable to elastic deformations calculated

TABLE I. NAILED JOINT FAILURE MODES

Test series	Gusset type	Test no.	Failure mode
A	Steel	9	Nail bending
		4	Lower chord tension at knot
		11	Nail bending
		25	Gussets torn at nail holes
		29	Gussets torn at lower chord nails
		1	Lower chord tension at knots
B	5-ply D. fir	8	Nail bending
		10	Nail bending
		22	Nail bending
		24	Nail bending, tearing splits at top chord
		26	Nail bending, tearing splits at top chord
		27	Nail bending
C	4-ply D. fir	5	Lower chord tension
		12	Heel gusset tension
		19	Nail bending
		20	Rolling shear in heel gusset
		28	Heel gusset tension
		30	Heel gusset tension
D	5-ply spruce	6	Heel gusset tension
		7	Nail bending
		17	Nail bending
		18	Nail bending
		21	Heel gusset tension
		23	Nail bending
E	4-ply spruce	3	Heel gusset tension
		13	Heel gusset longitudinal shear
		15	Heel gusset tension
		16	Heel gusset delamination
		2	Heel gusset delamination
		14	Nail bending

for the wood frames and the steel or plywood gussets. These adjustments were based on another Williot-Mohr diagram. Hydraulic piston travel D was registered by two linear potentiometers, one mounted each side of the specimen ridge joint. Pairs of potentiometer readings and corresponding time and load data were printed out at 1-min intervals by a data-logger (Kaye Digistrip II).

The vertical resisting force developed at the ridge joint of each test frame was indicated by a calibrated flat load cell (Strainsert Universal, Model FL25V-2SP) inserted between the hydraulic piston and a ball-jointed steel plate bearing on the top of the ridge joint. As with the deformations, the joint loads P were sensed indirectly at the ridge joint and converted to frame-to-gusset nail loads Y, using the following relationship:

$$Y = 0.1135\, P \tag{3}$$

where Y = frame-to-gusset load (kN/nail) and P = ridge joint load (kN).

The value of nail load Y in Eq. 3 was calculated from the number of nails per

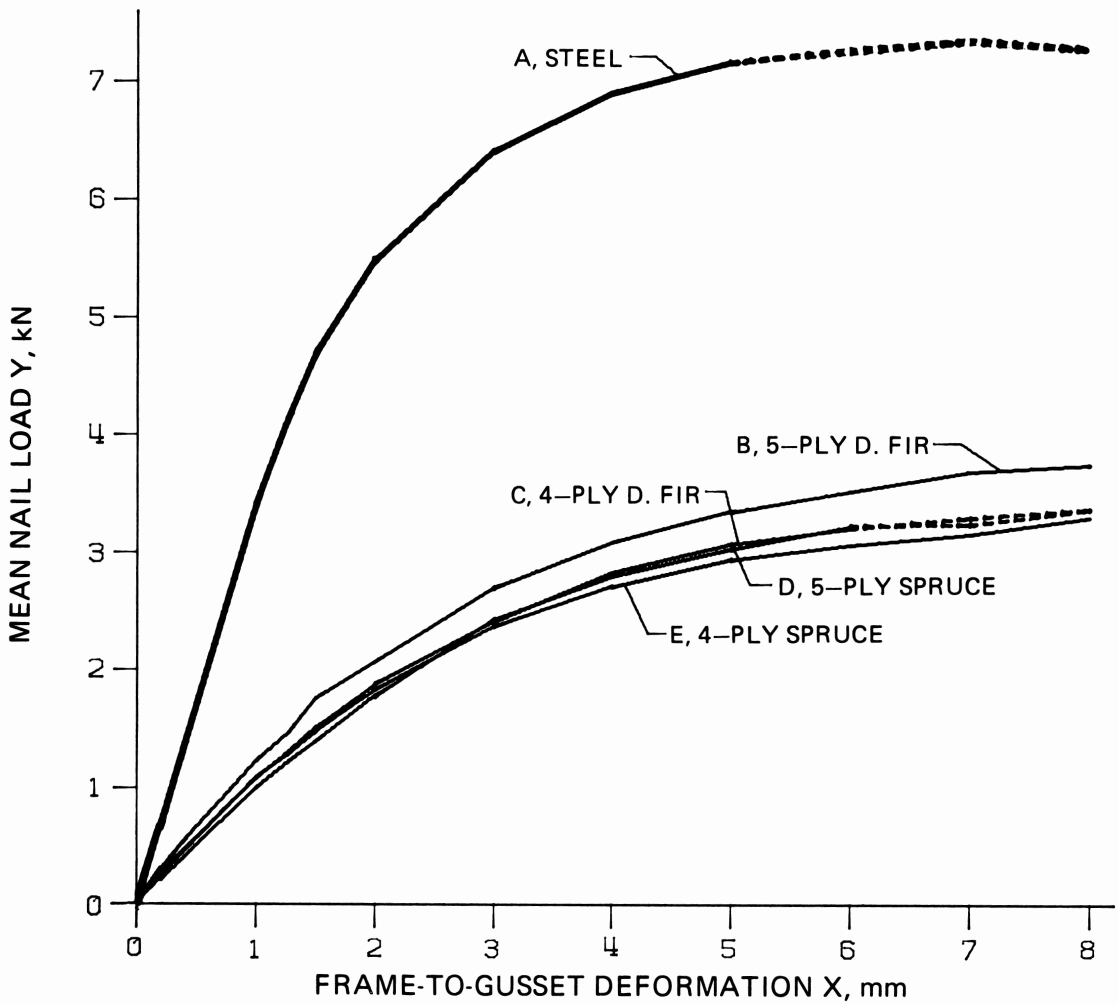

Figure 2. Mean nail load versus joint deformation, as tested at 1.91 mm/min deformation rate. Deformations (X) were not corrected for frame and gusset elastic strains.

connection and the geometry of the A-frame, with all connections considered as pinned. However, because the centroids of the nail groups did not coincide with the member centerlines, the effective slope of the upper chords was taken as 20° to the horizontal, not the 18.4° that would correspond with a 1/3 rafter slope.

During each test the data-logger was programmed to scan the test load and the paired potentimeter readings once each minute. The joint displacements of particular interest were $X = 0.38$ and 1.27 mm, corresponding, respectively, to nail design loads traditionally used for high human occupancy (HHO) and low human occupancy (LHO) farm buildings as defined by the Task Group on Farm Buildings (1983). Using the method of cubic splines for interpolation, the values of nail resistance Y corresponding to nailed joint displacements $X = 0.38$ and 1.27 mm were obtained. These 60 values (five tests, six replications, two slips) were then subjected to an analysis of variance.

The order of testing was randomized to eliminate bias due to operator experience, drying of wood, etc. The mode of failure (nails, gussets, frame, etc.) was noted.

RESULTS AND DISCUSSION

Statistical Analysis of Test Loads

The ultimate test loads in Table III were not statistically analyzed because of the complicating effects of other failures not related to the nailed joints (see Table I).

All the test data in the range of joint slip from 0 to 2 mm were analyzed. Within this limited range the departures from linearity were minimal, although there was occasional slight evidence of nonlinear behavior. Figure 2 shows mean performance curves for the five joint types, although (unlike the analyzed data) these curves were not adjusted for frame deformations.

The analysis of variance of nail resistance values corresponding to joint slips 0.38 and 1.27 mm showed that the steel connections (series A) were significantly more than twice as strong as all other tests, at both 0.38 and 1.27 mm slip ($P < 0.001$). As expected, the series A connections (steel gussets) were also stronger than the previous double-member connections (steel, and steel with five-ply D. fir) (Turnbull et al. 1981). This relationship was not statistically examined; however, the series A ultimate loads were lower than would be expected in relation to the previous steel and steel-plywood strengths.

Considering again the nail resistances at 0.38 and 1.27 mm slip, the five-ply plywoods produced significantly stiffer nailed connections than the corresponding four-ply plywoods ($P = 0.035$). Also, the D. fir plywood connections were significantly stiffer than those made with spruce plywoods ($P < 0.001$). There was no significant interaction between the wood species and the number of plies in this limited range of slip.

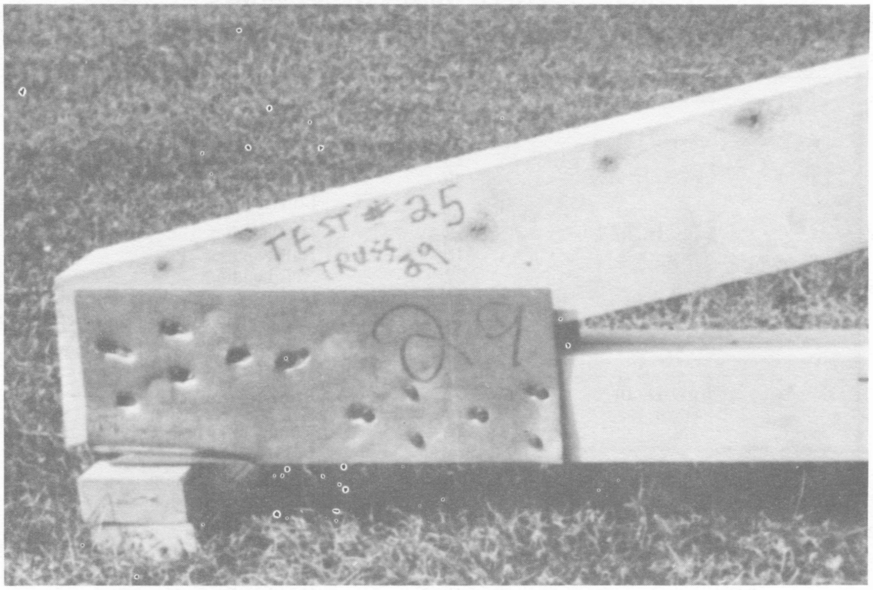

Figure 3. Series A steel tests typical failure, due to tearing elongation of the nail hole.

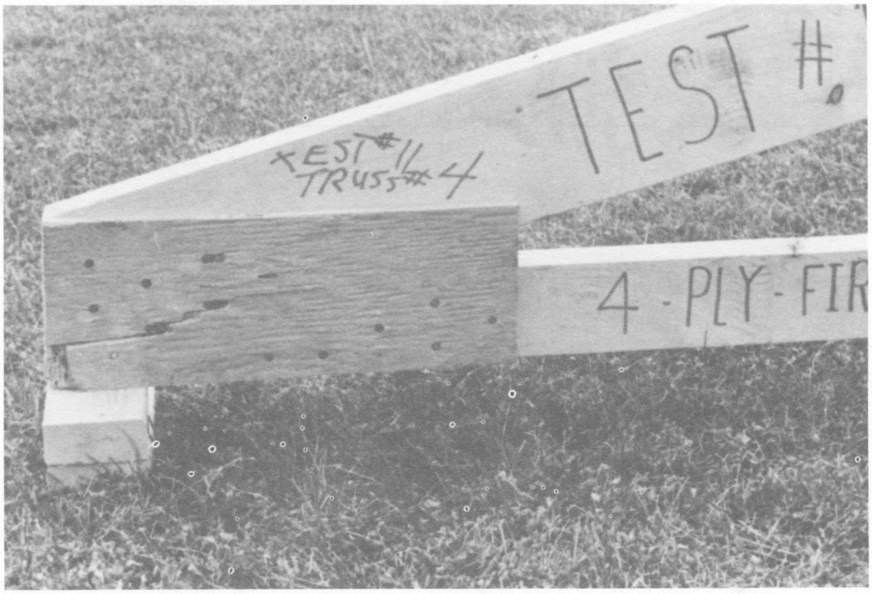

Figure 4. Series C four-ply D. fir plywood failure, a diagonal failure due to tension in the plywood gusset.

Failure Modes and Ultimate Nail Loads

Table I gives the failure mode for each of the 30 tests. Failures due to nail bending and gussets tearing at the nail holes were not abrupt (that is, the load reached a peak value, then declined from this peak value as deformation continued to increase). On the other hand, failures due to gusset or frame rupture were abrupt.

Series A steel tests showed a variety of failure modes; two were due to nails bending, two to lower chord tension (at knots) and two to elongated nail holes torn in the steel gussets (Fig. 3). Gusset tearing was more equally distributed between side and center gussets, as compared with previous tests (Turnbull et al. 1981) where all three gussets had been of equal thickness.

Series B five-ply D. fir plywood tests all failed in nail bending, at a mean load of 3.61 kN/nail; this was 49% of the mean failure load for the Series A steel tests. There was no apparent damage to the five-ply D. fir plywood gussets except

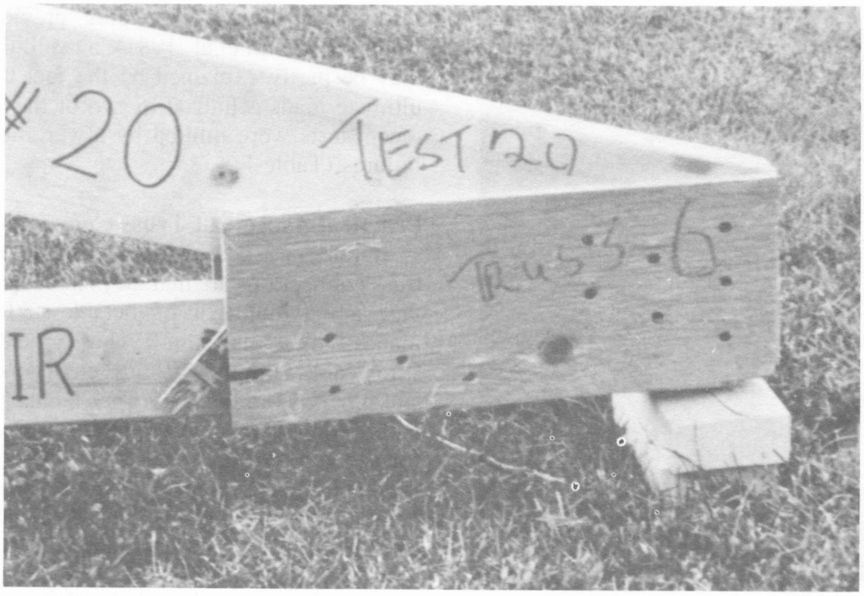

Figure 5. Another series C four-ply D. fir plywood failure, due to rolling shear delamination within the two vertical center plies.

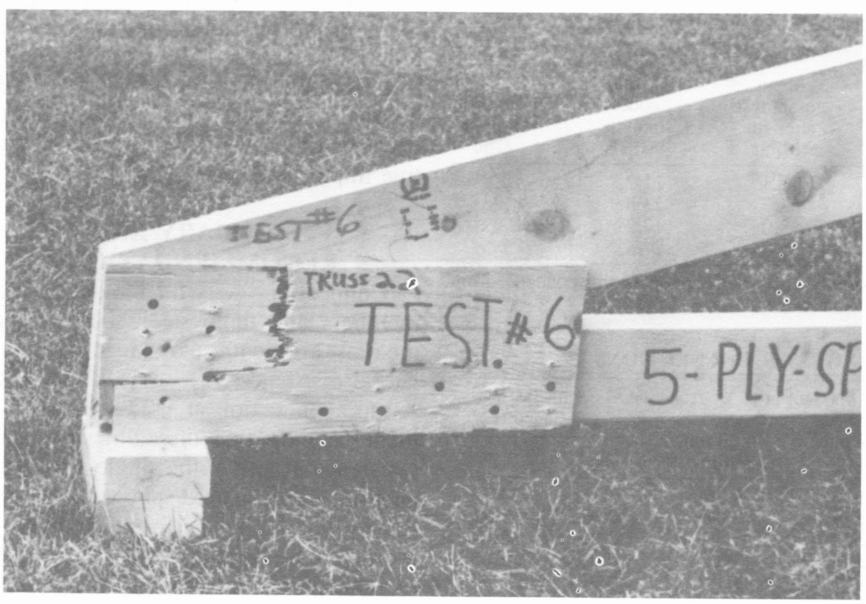

Figure 6. Series D five-ply spruce plywood, typical tension failure.

localized crushing at the nail holes.

In contrast to the five-ply D. fir, Series C four-ply D. fir plywood showed four plywood failures, one nail failure and one lower chord failure. The lower chord failure was not of interest to this experiment, but at 3.26 kN/nail it had little effect on the mean failure load of 3.27 kN. The failures shown in Fig. 4 (plywood tension) and Fig. 5 (plywood rolling shear delamination) were typical of this Series.

Series D five-ply spruce plywood had failure loads clearly superior to the four-ply D. fir but inferior to the five-ply D. fir. There were two plywood tension failures (Fig. 6 is typical) but there was no evidence of rolling shear plywood delamination as found in the four-ply plywoods, both D. fir and spruce.

Series E four-ply spruce tests showed four plywood failures and one nail bending failure, with the mean ultimate nail load about equal to that of four-ply D. fir. Figure 7 shows one of the plywood failures, in this case a longitudinal shear.

Plywood Failure Stresses Versus Specified Strengths

Using specified strengths and strength modification factors from the CSA 086 standard (Technical Committee on Engineering Design in Wood (TCEDW) 1984), plywood test stresses provide an interesting (if not conclusive) comparison.

For discussion purposes Table II shows "strength ratios" (column 7) for five-ply and four-ply plywoods. Strength ratio is defined here as the ultimate plywood test strength divided by the factored strength predicted by CSA 086 (TCEDW 1984). Strength ratio was calculated as follows: Mean ultimate gusset tension (column 4) was calculated from the mean ultimate load achieved in each test series and from the geometry of the A-frame test specimen assuming it behaves as a pin-jointed truss with an effective upper chord angle of 20° with respect to the lower chord. Mean gusset test strengths (column 5) were calculated from the forces (column 4) applied to the effective section area of the horizontal plies of the plywood gusset pairs (vertical plies were assumed to have no strength perpendicular to grain). The mean gusset test strengths included both tension and bending components because the gusset tension force was estimated to act 25 mm above the centerline of the effective plywood section area.

Factored code strength for D. fir plywood (column 6) was calculated for a pair of gussets, total depth 300 mm, based on Table 56, CSA 086 (TCEDW 1984). The specified strength capacity factors included $X_J = 0.85$ for unsymmetrical tension within each gusset, where gussets are applied to both sides of a joint (clause 7.4.2.4, TCEDW 1984). Factored code strengths for the spruce plywoods were calculated from unpublished specified strengths (obtained by personal correspondence, C. K. A. Stieda, Council of Forest Industries of British Columbia, North Vancouver).

There were no failures in the five-ply D. fir plywood, an observation that is consistent with the lowest test: code strength ratio of 2.0. The four-ply D. fir gussets failed in four of the six tests, again consistent with a higher strength ratio of 2.3. The spruce plywoods were again consistent, with only two plywood failures in the five-ply (strength ratio 2.2) but four failures in the four-ply (strength ratio 2.5).

It is interesting to note here that only 25 mm of eccentricity estimated for the gusset tension force adds a "moment" component that virtually doubles the calculated combined stress at the critical top edge of the gussets. Tension failures probably start at such locations of stress concentration and tear progressively from there, although such failures occur too abruptly to be so observed without special equipment.

Unit Lateral Resistance for Nailed Gusset Connections

For comparison and design purposes, Table III was prepared from the present

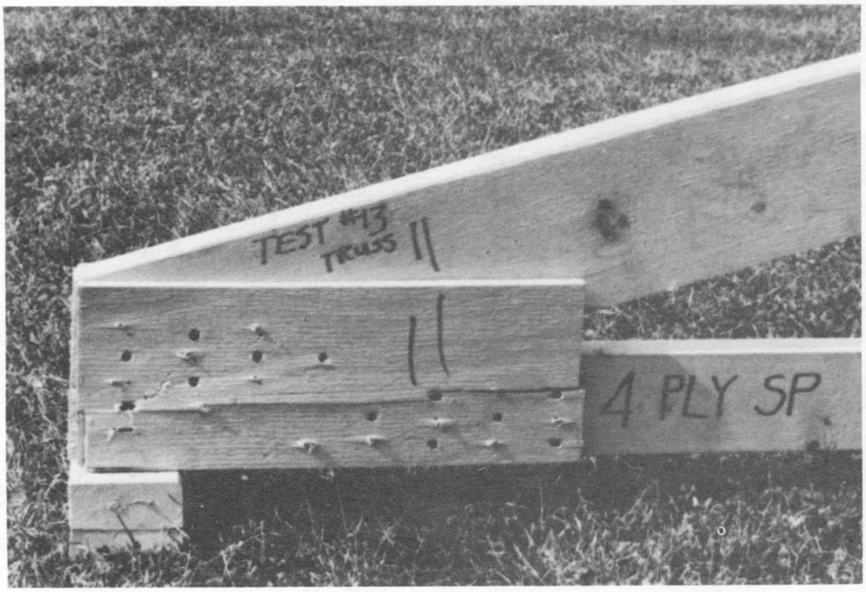

Figure 7. Series E four-ply spruce plywood failure, in this case a longitudinal shear.

TABLE II. PLYWOOD GUSSET STRENGTH COMPARISONS

1	2	3	4	5	6	7
Test series	Gusset type	No. of gusset failures	Mean ult. gusset tension (kN)	Mean gusset test strength (MPa)	Factored code strength (MPa)	Test/code strength ratio
B	5-ply D. fir	0	45.7	42.3	21.2	2.0
C	4-ply D. fir	4	39.4	54.7	23.8	2.3
D	5-ply spruce	2	41.0	38.0	17.2†	2.2†
E	4-ply spruce	4	39.6	55.0	21.8†	2.5†

†Based on unpublished specified strengths for Canadian softwood plywood (personal correspondence from C. K. A. Stieda, Council of Forest Industries of B.C., N. Vancouver).

TABLE III. RESISTANCE OF NAILED GUSSET CONNECTIONS

		Test load, kN/nail at			Unit lateral resistance (kN)	
Test series	Gusset type	0.38 mm slip	1.27 mm slip	6.00 mm slip†	N_s	N_u
A	Steel	**1.952**‡	**5.046**	7.36§	3.15	4.73
B	5-ply D. fir	**0.640**	**1.579**	3.61	0.99	2.25
C	4-ply D. fir	**0.520**	**1.528**	3.27	0.96	2.04
D	5-ply spruce	**0.525**	**1.575**	3.24	0.98	2.03
E	4-ply spruce	**0.477**	**1.457**	3.11	0.91	1.94
(1981)	5-ply D. fir	1.03	2.36	4.26	1.47	2.66
(1964)	5-ply D. fir	0.95	1.87	3.37	1.17	2.11
(1981)	Steel	1.41	4.14	8.07	2.59	5.04
(1981)	Steel, 5-ply D. fir	1.53	4.59	8.14	2.87	5.08

†Ultimate test load, with nail slip adjusted for elastic frame and gusset deformations (Eqs. 1 and 2).
‡Loads in boldface type were statistically analyzed. The standard error for comparing wood species is 0.044, and for comparing steel with wood species is 0.087.
§Some steel tests reached maximum test load before 6.00 mm slip occurred.

and several previous tests. For design of LHO farm buildings, nailed truss connections have been based traditionally on test loads recorded at 1.27 mm (0.050 inch) joint slip. The "service" lateral resistances (N_s) were the test loads per nail at 1.27 mm slip, factored by 0.625 to "normal" (10-yr) load duration. Similarly the "ultimate" lateral resistances (N_u) were the ultimate test loads at 6 mm nail slip, also factored by 0.625 to give "normal" load duration. The lateral resistance values N_s and N_u are used here as defined in Clause 10.8.4.1 of the "limit states design" version of the CSA standard (TCEDW 1984).

A general observation on the 1985 plywood nailed joint results in Table III is that both N_s and N_u design values are lower than previous results. On the other hand, the new N_s value for steel gussets is about 10% greater than those given by Turnbull et al. (1981), due probably to the increased thickness of the center steel gusset. However these new "Ultimate" and corresponding N_u values for steel are 7% lower than the corresponding previous values (Turnbull et al. 1981), a result that may be partly explained by the fact that ultimate loads achieved in two of the six recent tests were limited by lower chord failures (Table I).

Practical Aspects of Truss Connections

Important aspects of these field-nailed truss connections are that they should provide a maximum of predictable performance with a minimum level of carpentry skills, and seldom with any engineering site supervision. In terms of the number of nails required to connect truss members for a given roof area and load condition, the steel gusset system A is by far the best compared with systems B and C. Unfortunately the carpenters found it was not easy to drive the concrete nails through the steel gussets, particularly the thicker (1.22 mm) center gusset. There was no problem with nail blunting or bending, but hard blows with a club hammer were required to penetrate the steel. An alternative would be to predrill undersized pilot holes to make the nailing easier, but the extra labor probably makes this alternative impractical.

SUMMARY AND RECOMMENDATIONS

Connection A with doubled frame members and three steel gussets makes the most efficient use of nails. With a unit lateral resistance N_s = 3.15 kN/nail, the A-type connection at 1.27 mm displacement has more than three times the lateral resistance of any of the single-frame connections with paired plywood gussets. Part of this higher performance is due to more planes of shear (3 vs. 2), part is due to a thicker concrete nail (4.5 vs. 4.0 mm diameter) and part is due to steel gussets. On the negative side, the ultimate strength of the A-type connection was not up to expectations, and it was hard to drive the nails through the thicker center plate, even with a heavy club hammer.

Comparing the plywood gusset types, all using the 4.0 × 63-mm concrete nails, the traditional five-ply D. fir plywood (series B) proved clearly superior to the four-ply D. fir (C) and the spruce plywoods (D and E). The four-ply plywoods (C and E) both showed gusset failures predominating. Checking the plywood failure stresses against factored strengths predicted according to the wood design code (TCEDW 1984) tended to confirm the safety and consistency of the code.

For single-member trusses that make up the greater part of the CPS truss design family, no satisfactory substitute was found for the five-ply D. fir gusset material that has virtually disappeared from the market.

For double-member trusses, the triple steel gussets with a thicker center gusset performed very well, but these joints were difficult to assemble without predrilling.

Further research with thicker five-ply or seven-ply D. fir plywood gussets is recommended, for connecting single-member trusses. It is worth noting that 76-mm concrete nails would be compatible with the total thickness of two 18.5-mm plywood gussets and one 38-mm center member. Using 18.5-mm plywood would at least allow an adequate odd number of plies in the plywood lay-up and would better match the strength of the 38-mm frame members to be connected.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the major contributions by engineering students Scott Laking and Kevin McKague who built the test specimens and ran the performance tests for this project, and by technologist Don Lowe who supervised and carried out the laboratory work. Helpful comments and data by Dr. C. K. A. Steida, Council of Forest Industries of British Columbia, are also acknowledged.

REFERENCES

ANONYMOUS. 1978. Douglas fir plywood. CSA Standard 0121-M1978, Canadian Standards Assoc., 178 Rexdale Blvd., Rexdale, Ont.

MASSÉ, D. I. 1985. Design and analysis of multi-laminated nailed joint trusses. M.Sc. thesis, Faculty of Engineering, Carleton University, Ottawa, Ont.

TASK GROUP ON FARM BUILDINGS. 1983. Canadian farm building code. National Research Council of Canada, Ottawa, Ont. NRCC publ. no. 21312.

TECHNICAL COMMITTEE ON ENGINEERING DESIGN IN WOOD. 1984. Engineering design in wood (limit states design). CAN3-086.1-M84, Can. Stds. Assoc., 178 Rexdale Blvd., Rexdale, Ont.

TURNBULL, J. E., L. P. LEFKOVITCH, and D. M. LOWE. 1981. Multilaminated nailed truss connections. Can. Agric. Eng. 23: 113−119.

TURNBULL, J. E., D. I. MASSÉ, and D. J. BUCKLEY. 1983. First experiences with a new facility for testing wood-framed farm roof trusses, diaphragms and rigid frames. Can. Agric. Eng. 25: 101−109.

TURNBULL, J. E. and F. H. THEAKSTON. 1964. Analysis of plywood gusset connections with steel nails. EIC-64-BR and STR3, Division paper of the Eng. Inst. of Can. 2: 2, March.