MULTI-LAMINATED NAILED TRUSS CONNECTIONS

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To provide designs for heavy-duty wood truss connections suitable for the economical and useful 2.4-m truss spacing now frequently used in farm buildings, four multiple-shear nailed gusset connections were evaluated. The best all-round connection was a five-member system consisting of doubled 38-mm frames with a 0.95-mm galvanized steel gusset between frames and 12.5-mm Douglas fir plywood gussets on both outsides. Using ordinary 4 × 102-mm spiral nails driven from both sides, a design load of 2.41 kN/nail was determined, compared with 1.36 kN/nail for 4 × 64-mm concrete nails used in the traditional three-member single-frame joint used in existing Canada Plan Service truss designs.

INTRODUCTION

In Canada, factory-made wood trusses and rigid frames are most commonly connected by toothed press-plates of galvanized steel. This connecting method is fast and easily adapted to mass production methods, but it is not suitable for site-built structures assembled by small contractors or farmers. On-site truss building is more common in rural areas remote from prefabricating plants. Here hand nailing has retained a place in farm building roof truss construction, so that the development of more efficient hand-nailing methods is justified.

The Canada Plan Service (CPS) has distributed a full range of designs for hand-nailed wood trusses based on nailed connections developed by Turnbull and Theakston (1964). These connections used single, 38-mm thick frame members sandwiched between two 12.5-mm plywood gusset plates and nailed from both sides with special thick-shank hardened spiral 'Truss Gusset' nails, loaded in double shear. The nails were originally made by the Steel Co. of Canada; similar nails have since been made by other Canadian nail manufacturers.

CPS gable roof truss designs (prepared during 1965–1968 and again during 1972–1974) specified this double-shear, three-member nailed connection made with Truss Gusset nails. Recently, provincial extension engineers have reported increasing difficulties in obtaining the special nails, so the 1980 CPS metric trusses as well as the CPS gambrel roof arch designs (Jackson and Turnbull 1979) have specified 4 × 64-mm concrete nails which are generally available. Compared to 'Truss Gusset' nails, the number of concrete nails per connection should be increased about 15%, a minor inconvenience.

Evolution of farm building design has trended to the framing of walls with poles spaced at 2.4 m supporting heavy-duty roof trusses at the same 2.4 m spacing. This increased truss spacing to 2.4 m does not change the total amount of wood required to frame a clear-span roof for a given span and snow load, but it doubles the mass of each truss to be hoisted, halves the number of hoistings, and eliminates the heavy laminated wall plate beam formerly used to support trusses arriving between the poles. A further advantage is that each truss can bear directly on end-grain wood at the bottom of a notch cut into the top of its corresponding wall pole; this usually gives adequate bearing area, which is less easily achieved with trusses bearing perpendicular to the grain of a horizontal wall plate or plate beam. Roof purlins are usually placed on edge to handle the increased purlin span from truss to truss; these purlins on edge need nailing-clips or wood cleats to prevent roll or uplift where they are fastened to the trusses.

The increase to 2.4-m truss spacing results in very high truss joint loads, namely two or four times the corresponding joint loads for trusses spaced at 1.2 or 0.6 m, respectively.

Turnbull and Theakston (1964) found that a symmetrical plywood-on-frame, three-member nailed joint resulted in an allowable nail load of twice the single-shear nail load; this principle can likewise be extended to take advantage of any attainable number of shear planes. We proposed that several multi-laminated joint designs could be evaluated, with nail lengths chosen to penetrate the total thickness of the joint and to take advantage of ordinary nails readily available in any Canadian hardware store or lumber yard.

THE EXPERIMENT

We compared three new joint designs with the original three-member CPS-type truss joint. Figure 1 gives details of the four joint types tested. Type A is the original CPS three-member design using 4 × 64-mm hardened concrete nails and two gusset plates of 12.5 mm exterior sheathing Douglas fir plywood; other types B, C and D all use two laminations of 38-mm spruce frame and three gusset plates.

Type B is a five-member joint with three gussets of galvanized sheet steel, 20 gauge (Manufacturer's Standard Gauge), or 0.95-mm thick including galvanizing, and nailed with 4.5 × 76-mm concrete nails. In this type, each nail develops three shear planes (penetration into the third steel gusset is not significant).

Type C is a compound five-member joint with two outside gussets of 12.5-mm plywood and one center gusset of 0.95-mm galvanized steel. This gives a total joint thickness of 102 mm which conveniently takes advantage of 4 × 102-mm common spiral nails, giving four shear planes per nail. The first series of type C tests (called C 3, 1979 tests, and C 6, 1980 tests) used hexagonal-spiral nails by Sivaco Wire and Nail Co., Marieville, Quebec. Another series (called C 5, also tested in 1979) used nails of the same nominal size, but with the four-sided 'Ardox' spiral form, by the Steel Co. of Canada, Hamilton, Ontario. The thickness of steel gussets in this series (and type B as well) was determined by a preliminary hand-nailing test in which steel thickness was progressively increased until difficulties developed with nail bending and sheet penetration. Then we reverted back to '20 gauge' which posed no particular nailing difficulties.

Type D is another five-member joint, with three gussets of 18.5-mm Douglas fir plywood. This plywood thickness was chosen to utilize fully the common smooth-shank 4.7 × 127-mm (nominal 5-inch) nails. Spiral-shank nails would have been preferred here as well, but Ottawa retail stores did not stock them in this length. Assuming similar supply dif-

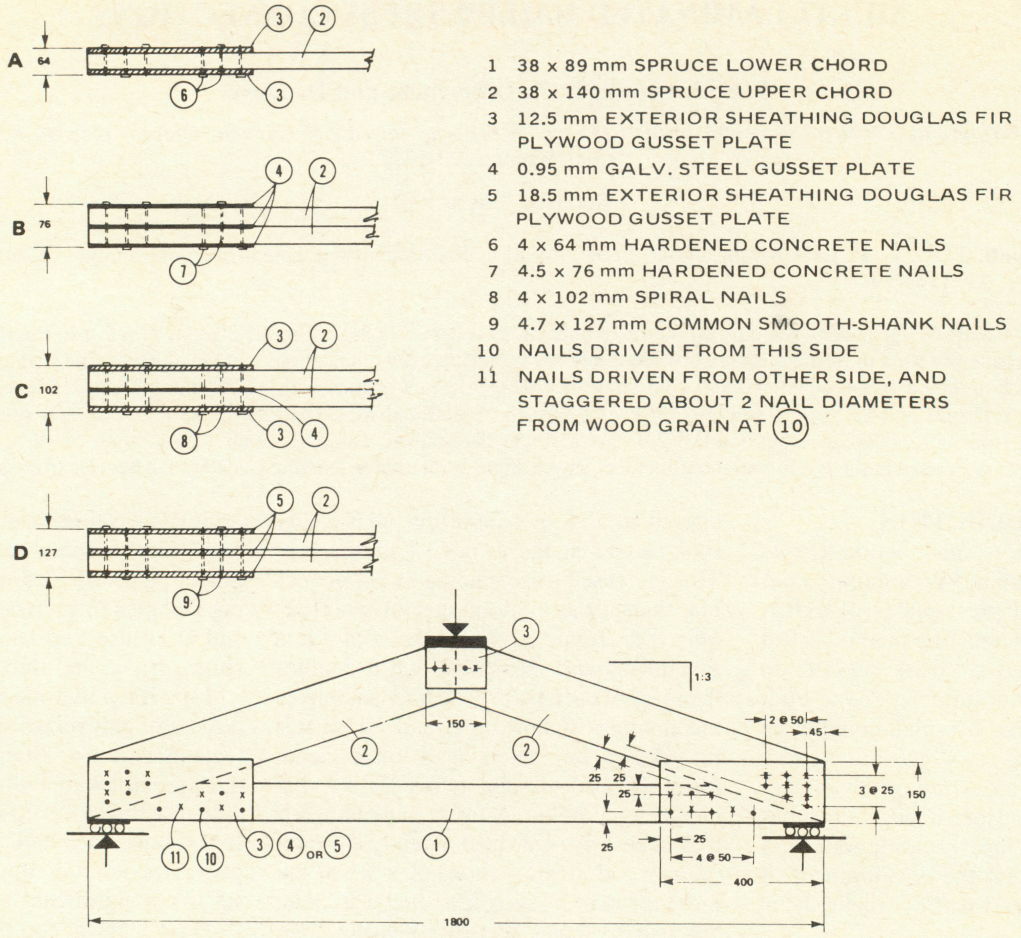

Figure 1. Details of test specimens to compare four types of multiple-shear nailed connections. Plan views A through D give details of four different treatments compared.

ficulties could apply elsewhere, it seemed advisable to use the more readily available smooth-shank type nails.

All joint types used the same A-frame simulated truss design (Fig. 1). It was felt that this would impose a more typical combination of shear and rotational effects on each joint assembly and would therefore produce more realistic nail loads for designing laminated trusses. However, this complicated the computation of actual joint displacements, as compared with the simple compression-shear tests used previously (Turnbull 1964). It is probably true that the simulated trusses used in this experiment are more suitable for applied research leading to truss design standards, than for fundamental investigations into nailed joint characteristics.

Determination of Nail Loads for Design of Structures

The Canadian Farm Building Code (Standing Committee on Farm Buildings 1977) defines a 'low human occupancy' farm building (hereafter called LHO), and allows certain relaxations in the allowable connection loads and material stresses used for design of such buildings.

Turnbull and Theakston (1964) determined design nail loads based on the test load supported at 1.27 mm gusset-to-wood displacement for LHO and 0.38 mm for HHO (high human occupancy) which includes most other wood-frame buildings. These deformations are traditional and somewhat arbitrary (being originally 0.050 and 0.015 inch), but they tend to give acceptable structural rigidity and factors of safety and the two classes of wood-frame buildings considered in this context. Therefore the load values about joint deformations of 1.27 and 0.38 mm were used here.

Load duration is another factor which must be considered. For accelerated testing, it was considered that a load test of several minutes duration could be related to a 'normal' (10 yr) load duration by a multiplying factor of 0.575, based on a U.S. Forest Prod. Lab. report R1916 (Anonymous 1951).

Test Apparatus

In order to duplicate as closely as possible the joint displacement velocity of 0.32 mm/min used previously, a special testing frame was constructed using a large-diameter hydraulic cylinder for loading. From the geometry of the A-frame test specimens (Fig. 1), a required hydraulic piston velocity of 1.91 mm/min was calculated. To achieve this low piston velocity, the hydraulic cylinder diameter was oversized (152 mm), and the remote hydraulic power unit was designed around a special small-diameter adjustable-stroke hydraulic piston pump. The power unit was hose-coupled to the testing frame during calibration and load tests, and was calibrated for the required ram velocity while under simulated load (truck springs). The power unit was hose-coupled to the testing frame during calibration and load tests.

To record test loads and displacements continuously, a flat load cell (Strainsert, Model FL25U-3DP) and a displacement transducer (R.I. Controls, model 4046) were both coupled through a strain gauge transducer input module (Daytronic, type 93) to an X-Y recorder (Hewlett-Packard Mosely, 7004A), reading kilograms load (Y) and millimetres displacement (X at the 'ridge' joint.

A further problem was that test displacements were read vertically at the top center joint of each A-frame, whereas the displacements required for truss design correspond to those within the two heel joints, and were more or less parallel to the grain of the wood frames. This conversion involved corrections for the geometry of the A-frame specimens (similar to calculations previously mentioned for correcting the hydraulic ram velocity), as well as corrections for elastic deformations in the 38-mm frame members and the gussets. Since the objective was to extract the interior joint displacements (frame-nail-gusset, or vice versa), a Williot displacement diagram was drawn for each test series. From these diagrams a set of correction equations was prepared, from which parallel-to-grain joint displacements were calculated from those vertical displacements recorded at the top joint of each specimen. The top joint was a well-fitted compression joint with frames bearing directly on each other and on the top loading plate, so this part was assumed to allow relative member rotations, but no relative linear displacement within the joint.

The nail deformation correction equations were as follows:

Series A	$X = 0.1763\,(D - 0.00209P)$	(1)
Series B	$X = 0.1763\,(D - 0.000989\,P)$	(2)
Series C and D	$X = 0.1763\,(D - 0.00104\,P)$	(3)

where X = frame-nail-gusset deformation (mm); D = displacement at ridge joint (mm); and P = load at ridge joint (kg).

In equations 1, 2 and 3 above, the 0.1763 factor corrects for the truss geometry, and the terms $(-\ 0.00209\ P)$ etc. subtract the accumulated elastic effects in the truss members and gusset plates.

The load equations to correct for truss geometry and simultaneously to convert from kilogram to kilonewtons was:

$$Y = 0.00164\,P \tag{4}$$

where Y = load per nail (kN) and P = load at ridge joint (kg)

Preparation of Test Specimens

A supply of 38 × 235 × 4200-mm planks (no. 1 grade S-P-F) was purchased. This plank size was chosen to allow all three frame parts for each 'truss' to be cut from the same plank. Only spruce planks were selected from a shipment classed as 'spruce-pine-fir' (S-P-F), and these were subsequently re-sorted to ensure that each test type (A, B, C and D; Fig. 1) included a wood density distribution curve similar to the other series and typical of spruce. Densities ranged from 0.330 to 0.426, with a mean value of 0.373 g/mL, or 373 kg/m^3.

The wood stock was stored 'green' in a conditioning chamber controlled at 21°C, 90% RH, until it approached 24% moisture (dry basis). The plywood gusset plates were cut from dry stock as received from the dealer, and kept dry prior to being assembled. The spruce frame parts were cut from the 24% moisture-conditioned stock and assembled. Trusses were then returned to the chamber at 21°C, 70% RH for about 6 wk at which time the wood approached 14% moisture.

Moisture and wood density were based on standard wood-testing procedures as given in the following formulae:

$$M = \frac{W - Wd}{Wd} \times 100 \tag{5}$$

where M = moisture content (%); W = mass of specimen at test condition (g); and Wd = mass of same specimen oven dry (103°C) (g).

$$G = \frac{Wd}{V_i - V} \tag{6}$$

where G = density (g/mL); V_i = indicated volume of water displaced by wood at 'test' condition (mL); and V = water volume absorbed into specimen (mL).

The terms V_i and V in Eq. 6 need explanation. Each wood specimen was dipped briefly into a graduated cylinder of water, the rise in the water column was recorded as V_i, then the specimen was removed. The absorbed water volume V was then calculated by the difference between the initial and final water levels recorded before and after submerging and removing each specimen.

Testing

After the conditioning period, trusses with spruce frame members approaching 14% equilibrium moisture were removed one at a time from the chamber and immediately put on test. Each truss was positioned in the testing frame and loaded vertically at a uniform hydraulic ram velocity of 1.91 mm/min, calculated to give a heel joint displacement rate of 0.32 mm/min. Ridge joint vertical load and vertical travel were simultaneously traced on the X-Y recorder. It was necessary to break the load/displacement trace during each test and to switch recorder scales in order to record the critical early test stages with sufficient accuracy. After testing, corresponding X and Y values were read and tabulated from the recorder tracings and subsequently corrected (see Eqs. 2–4) to give net relative frame-nail-gusset displacement, X (mm) and load per nail, Y (kN). The statistical analysis involved 17–23 paired observations per truss, with emphasis on the region of particular interest, that is 0 to 1.5-mm net displacement.

Each truss was loaded to abrupt failure, or to the point where load peaked and started down, or to a maximum ram travel approaching 40 mm. It was then removed from the test frame, examined to determine the principal failure mode, and sectioned to remove small samples of the spruce frame for final determination of test wood density and moisture content. In total, 40 trusses were tested; five each of series A, B, C3 and D (in 1979), and 10 each of C5 and C6 (in 1980).

ANALYSIS OF RESULTS

Two main objectives in this analysis were (a) to decide which method provides the most suitable connection for hand-nailed trusses; and (b) to estimate the strength of each connection method, especially at 0.381-, 1.27- and 6.0-mm displacement. Loads (strength) at 6.0-mm displacement were considered as 'failure' loads and are only used in this study as indicators of the safety factor, by comparing them with 'design' loads obtained at 0.381- and 1.27-mm displacement.

Since direct observations of nail load could not be made at these net displacements, the usual statistical approach was to obtain a parametric model of the relationship. Using the proportional hazards model of life testing, an equation of the following form was postulated for each truss:

$$Y = a + b \log (X + 1) + \text{error}$$

where a and b are constants to be determined, and X has been increased by unity to permit zero deformation in the logarithmic transformation. The use of log $(X + 1)$ yielded a scale in which the range of values among the replicates within each treatment were effectively independent of the values of Y and log $(X + 1)$, and this transformation was retained for all subsequent analyses. This series of models was fitted, yielding the pooled squared deviations given in Table I. For each of the six treatments considered above, at least 92% of the total sums-of-squares (SOS) within treatments were accounted for, indicating a good overall fit.

TABLE I. SUMS OF SQUARED DEVIATIONS, $\Sigma\,(Y_i - \hat{Y}_i)^2$, FOR VARIOUS GROUPS OF THE DATA; 903 OBSERVATIONS CONSIDERED

Assuming	Mean	Parametric regression	(No. of parameters)	Kernel regression
One population	6803.744	877.945	(2)	742.961
Six treatments	6081.029	252.440	(12)	203.158
40 tests	6054.323	167.905	(80)	77.596

The effects of wood density and moisture content at time of testing, used as covariates, were also examined; since there were no significant relationships between these factors and the observed nail loads, their contributions have been omitted.

The type-D tests showed that the 4.7-mm smooth-shank nails made the stiffest joints within the range of safe nailing loads (see Fig. 2), but the joints, when sectioned after testing, showed excessive splitting of the spruce frame members (see Fig. 3D). The type D connection was therefore discarded as unsuitable for unsupervised farm building construction.

Series C 3, C 5 and C 6 (4 × 102-mm spiral nails, one steel and two plywood gussets) were very similar and can be considered equally 'best'. However, a plot of expected Y versus its residual from the observed values indicated that there were serious systematic (non-random) departures from the model (especially important in the region of lower loads) thus suggesting that it is not precise enough for predicting loads for design in spite of the apparently good general fit. This is probably due to the complex nature of a nailed

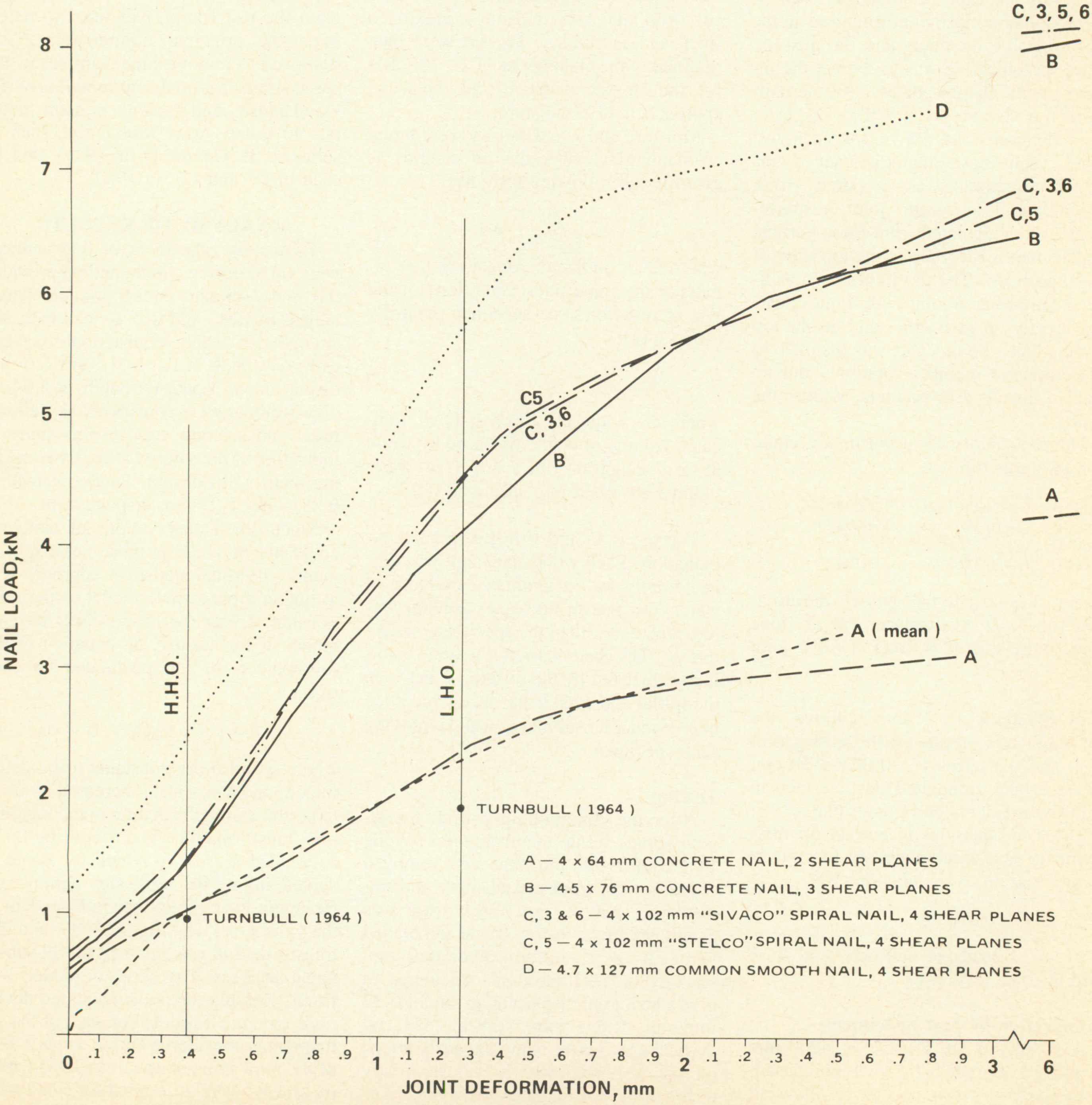

Figure 2. Expected load/deformation curves for four different nailed joint treatments.

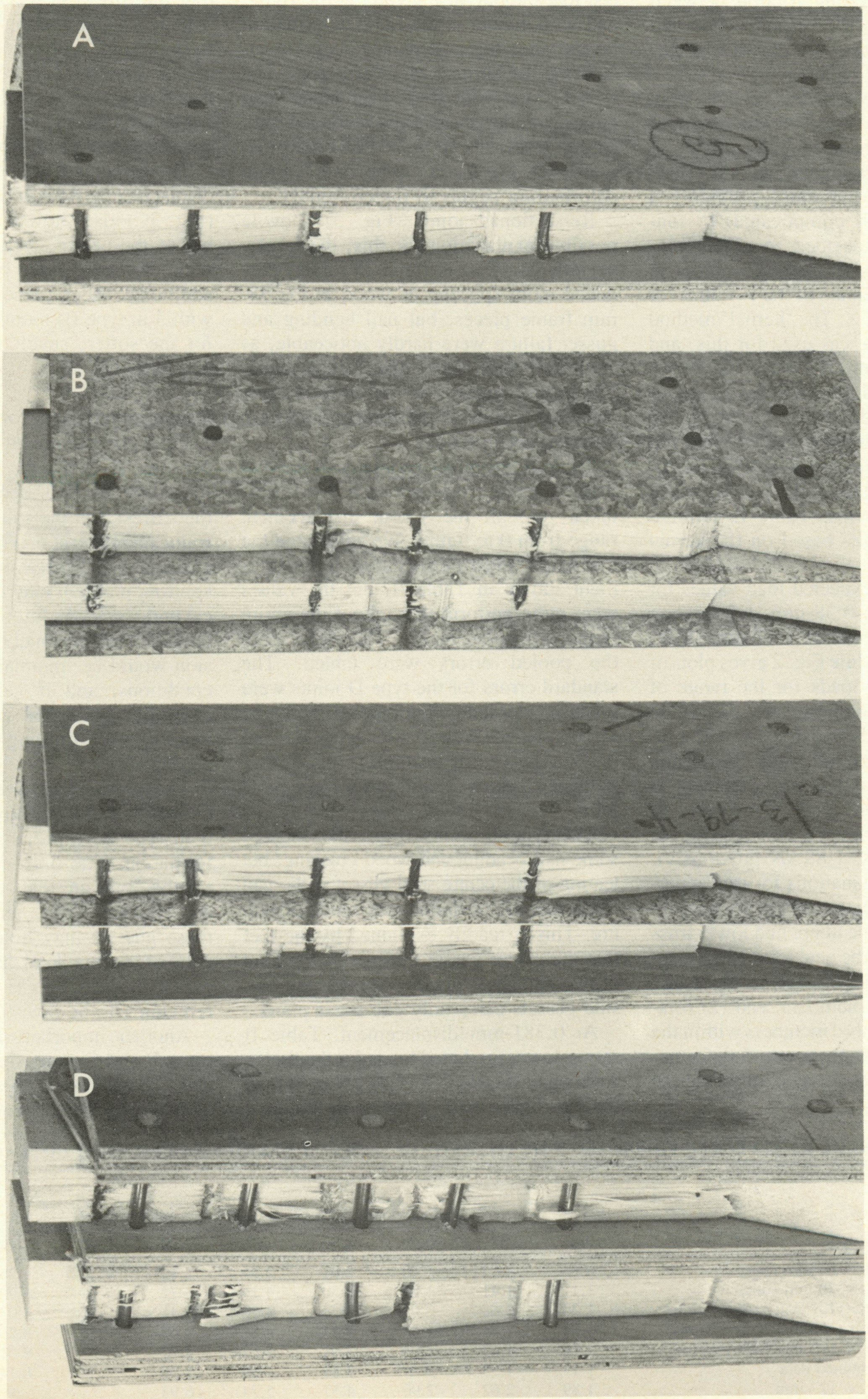

Figure 3. Four nailed joint treatments, after loading to failure; lower chords were split away to show modes of failure.

joint. Nailed joint deformation includes the accumulative effects of elastic and plastic nail bending, wood crushing, steel tearing, and increasing friction between the various layers of plywood, wood and steel as the interfaces are drawn together by increasing nail bending.

Several other parametric models were considered, some based on the supposed dynamics of the test specimens, others essentially just polynomials, and yet others taken piecewise, but those providing a good fit were very complicated. In consequence it was decided to use a non-parametric regression method in order to obtain a direct estimate of the Y-values for any given value of X. The 'kernel' method (Rosenblatt 1969) was used for this, and Epachenikov's optimal kernel (Epachenikov 1969) was adopted for the computations. Resulting differences between the computed and observed values were not only smaller than in the parametric model (Table I), but also there were no apparent systematic departures. Table II summarizes the test results based on the kernel method.

Table II gives expected load values for all joint types at 0.38-mm deformation (HHO buildings) and at 1.27 mm (LHO farm buildings), while Fig. 2 gives plots of the expected nail loads for the range of deformations considered.

Comparing the Series A loads with previous work (Turnbull and Theakston 1964), Fig. 2 shows close agreement at 0.381 mm displacement. However, at 1.27 mm this recent work produced higher loads (2.36 kN vs. 1.88 kN previously). Two possible explanations for this apparent increased stiffness are (1) friction developed between the upper and lower truss chord members at the heel joint when reaction forces increase enough to close any gaps between the members; and (2) rotation of the upper chord members within the heel joints at more pronounced deformations. These effects are also applicable to full-scale trusses.

Joint types B and C performed about equally and, compared to the traditional three-member joint (type A), produced test loads about 40% greater at the HHO, and 95% greater at the LHO deformation levels, showing a clear advantage to increasing the number of shear planes. The type D joint with four shear planes and larger, smooth-shank nails produced even greater test loads up to the LHO deformation level, but the curve beyond 1.5-mm deformation begins to flatten rapidly.

Re-sectioned joints (Fig. 3) show a possible explanation for this; the larger, smooth-shank nails (type D joints) produced more splitting of wood in the 38-mm frame pieces, but nail bending and gusset failure were hardly noticeable, as compared with A, B, and C type joints. The type D connection was therefore discarded as unsuitable for unsupervised farm building construction.

Table II also gives estimates of the nail load standard errors for each test series, taken through the whole displacement range from 0 to 6 mm. We calculated the nail load standard errors at each displacement shown in Table II, but the values were quite uniform throughout the entire range of displacements. Therefore, only the pooled errors were tabled. The standard errors for the type D joints were much greater than other types (0.503 type D, compared with 0.125 type A, and 0.174 type C, for example); this further indicates lack of reliability for the type D joint.

The type B joints (Fig. 3B) showed some nail bending and wood crushing, but tearing of the centre steel gusset at the nail holes was the predominating mode of failure. This is readily explained by the fact that the center gusset must carry the load of two nail shear planes, whereas the outside gussets carry only one shear plane.

At 0.381-mm displacement, Table II shows lower predicted loads for the Stelco Ardox nail (Series C 5, 1.41 kN/nail) than for the Sivaco nail of the same nominal size (Series C 3, and C 6, 1.68 and 1.47 kN/nail, respectively). One possible explanation is that the Stelco nail can penetrate the critical steel center gusset with its 'square' shank bearing at random angles from 0° to 45° with respect to the direction of the applied load. Square nails may therefore present a smaller projected bearing area on the steel gusset than the corresponding hexagonal Sivaco nail form.

This effect was not obvious at 1.27-mm displacement. At failure, the type C joints (Fig. 3C) also showed considerable tearing of the centre steel gussets combined with obvious nail bending, but the additional shear plane (4, as compared with 3 in type B) more than compensated for the softer, smaller-diameter nails in treatment C.

Recommendations for Design

From the standpoint of observed loads per nail at 0.38-mm and 1.27-mm deformation, the five-member type D joint is the stiffest, but the splitting of the spruce frame members at maximum load presents a disquieting aspect; one wonders if this splitting problem may be even more critical under real conditions where farmers or unskilled laborers nail the joints. Supervision would be minimal under these field conditions, and if nail rows were not carefully zig-zagged with respect to the wood grain, splitting failure could be significant.

The next best performing joint was type C, which was equal to or slightly better than type B, especially at 1.27-mm deformation. The only disadvantages seen for this joint are that two types of gusset are required (steel and plywood), and these gussets may hold disproportionate parts of the total load due to their different elastic moduli. This is an area of interest remaining to be investigated.

Another important advantage of both types B and C over type D is that types B and C have almost no space between the wood frame members of the truss; this can

TABLE II. SUMMARY OF EXPECTED PREDICTED NAIL LOADS (Y, kN nail) FOR VARIOUS NAILED JOINT DISPLACEMENTS (X, mm)

Test series	Nail type	Expected nail loads (kN/nail) at displacements (mm) of								Pooled standard errors
		0.25	0.381†	0.50	1.00	1.27‡	2.00	4.00	6.00§	(± kN/nail)
A	4.0 × 64 mm concrete	0.86	1.03	1.20	2.01	2.36	2.89	4.16	4.26	0.125
B	4.5 × 76 mm concrete	1.12	1.41	1.81	3.50	4.14	5.72	7.81	8.07	0.383
C 3 (1979)	4.0 × 102 mm Sivaco spiral	1.34	1.68	2.01	3.56	4.20	5.41	7.36	8.03	0.361
C 5 (1980)	4.0 × 102 mm Stelco Ardox	1.04	1.41	1.89	3.83	4.57	5.69	7.56	8.26	0.277
C 6 (1980)	4.0 × 102 mm Sivaco spiral	1.10	1.47	1.94	3.91	4.67	5.75	7.47	8.01	0.272
C 3, 5, 6	Combined data	1.17	1.53	2.01	3.89	4.59	5.69	7.54	8.14	0.174
D	4.7 × 127 mm	1.89	2.62	2.90	4.57	5.74	6.93	//	//	0.503

†X = 0.381-mm displacement traditionally used to obtain nail design loads for high human occupancy buildings.
‡X = 1.27-mm displacement used for low human occupancy farm buildings.
§X = 6.0 mm arbitrarily chosen to indicate joint failure.
//Several test specimens in this series failed by premature splitting in the wood frames, preventing full development of the nails.

provide some added buckling resistance for the doubled compression members of wood trusses, especially if the paired frame members are nailed together at intervals between the joints.

Table III gives recommended design loads based on the predicted 'most probable' loads in Table II and adjusted to 'normal' load duration by a multiplying factor of 0.575, as discussed previously. For the type A joint (as used in current CPS truss designs), it appears that allowable loads now used can be increased slightly, based on this new work which presumably simulates more closely the extra load resistance obtained when members are rotated as well as translated with a larger truss joint. The only truss connections to which this may not apply are those loaded in simple tension (lower chord of a roof truss, for example).

Safety factor is another consideration. Using the Table III design loads for HHO buildings, safety factors in the range of 4.15-5.73 indicate very low risk of failure, these designs being based primarily on joint stiffness. However, farm building designers using the LHO nail loads in Table III might prefer a slightly increased safety factor, say 2.0. With type C joints, for example, an increased safety factor of 2.0 corresponds to an LHO design load reduced from 2.41 to 2.30 kN/nail.

Two additional important items relating to Table III should be noted here. (1) In no case should the LHO design loads be further increased by 25% as allowed for LHO farm buildings in the Canadian Farm Building Code (1977), since this relaxation of design requirements was already done by increasing the allowed joint deformation. (2) For design of wood roof trusses largely determined by snow load (2 mo duration), the National Standard of Canada Can 3-086-M80 (1980) allows an increase of 15% in allowable nail loads as compared to 'normal' load duration; this increase is applicable here.

Design loads for type D joints were not included in Table III because of expected dubious field performance due to excessive splitting of the spruce frame pieces.

SUMMARY

Four types of multi-laminated nailed joints intended for farm-assembled wood frame roof trusses and other building frames were evaluated. These included re-evaluation of the traditional three-member CPS-type truss joint (plywood-frame-plywood, with 64-mm special concrete nails driven from both sides).

Additional evaluations were five-member joints using two 38-mm frame members alternating with three gusset plates. Increasing the number of nail-shear planes and the diameter (bending stiffness) of the nails were both found to be effective in making stronger, more rigid joints.

From several standpoints, a compound five-member joint is considered most suitable for connecting heavy-duty farm building trusses designed primarily for wider truss intervals (2400 mm) and heavier roof snow loads. This joint consisted of a 12.5-mm fir plywood outer gusset, 38-mm spruce frame, 0.95-mm galvanized steel center gusset, 38-mm frame, and a second 12.5-mm plywood outer gusset. Nailing from both sides with readily-available 4 × 102-mm (nominal 4-inch spiral-shank nails) completes the assembly.

For LHO farm buildings the design load for this connection is 2.41 kN/nail, compared with 1.08 kN/nail formerly used for the traditional CPS three-member joint made with 4 × 64-mm concrete nails. For designers who prefer a minimum safety factor of 2.0, this design load could be reduced to 2.30 kN/nail.

There may be situations where overall joint thickness should be limited to 79 mm, or where it is not convenient to use two different gusset materials (plywood and steel); for these situations, a five-member joint using three identical steel gussets and shorter 4.5 × 76-mm (nominal 3-inch) concrete nails performed almost as well as the compound plywood/steel gusset system.

Whenever very high joint forces occur (due to wider frame spacings, longer spans or heavier roof loads), doubled frame members can provide the necessary section properties, and the double-member system can frequently be less costly than a single larger member. Furthermore, doubled frame members offer the possibility of halving the total number of nails to complete the roof system, through use of multiple-shear nailing.

Where drawings have been prepared to take advantage of structural economies based on trusses spaced at 2400 mm, some builders may still prefer to use factory-prefabricated trusses. In such cases a simple alternative is to nail-laminate factory press-plate trusses together in pairs. This eliminates the need to redesign the truss-supporting structure to accommodate trusses at closer spacings (1200 mm, 600 mm, etc.), and it halves the number of trusses to be hoisted and secured to the wall system, a considerable saving of expensive crane rental time.

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TABLE III. DESIGN LOADS ('NORMAL' LOAD DURATION) AND CORRESPONDING SAFETY FACTORS (SF) FOR MULTI-LAMINATED NAILED TRUSS CONNECTIONS

Test series	Nail type	HHO design†		LHO design‡		Failure loads (kN/nail)
		kN/nail	SF	kN/nail	SF	
Turnbull	4.5 × 66 mm truss gusset	0.55		1.21		
(1964)	4.0 × 64 mm concrete	0.55		1.08		
A	4.0 × 64 mm concrete	0.59	4.15	1.36	1.80	2.45
B	4.5 × 76 mm concrete	0.81	5.73	2.38	1.95	4.64
C	4.0 × 102 mm spiral	0.81	5.68	2.41	1.91	4.60

†High human occupancy as defined in CFBC (1977), based on displacement X = 0.381 mm. Table II predicted loads X 0.575 (load duration factor).
‡Low human occupancy farm buildings, based on displacement X = 1.27 mm. Table II predicted loads times 0.575 (load duration factor).
§Based on displacement X = 6 mm.