PERFORMANCE OF SINGLE-MEMBER VERSUS DOUBLE-MEMBER GABLE ROOF TRUSSES

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Contribution no. 1-511, received 4 January 1984, accepted 10 October 1984


Two Canada Plan Service wood-frame gable roof truss designs for 9.6-m clear building span were load tested for structural performance under simulated snow loads. One truss design (intended for spacing at 1.2 m) used single frame members connected with two plywood side plates and 64-mm concrete nails; the other (M-9220, intended for spacing at 2.4 m) used double frame members, two plywood side plates and one center galvanized steel plate, with 102-mm common spiral nails. In all other respects their dimensions were similar. Mean failure load for the single-member trusses was 6.54 kN/m² of horizontal roof surface or 2.72 times design load. Mean failure load for the double-member trusses spaced at 2.4 m was 5.50 kN/m², or 2.29 times design load. Gussets of four-ply Douglas fir plywood (instead of traditional five-ply) contributed to 6 of the 10 truss failures.

INTRODUCTION

Based on multi-laminated nailed truss connections (Turnbull et al. 1981), the Canada Plan Service (CPS) design center has been developing a series of double-member wood roof trusses designed to be spaced at 2.4 m. Previous loading tests on one set of these double trusses (Turnbull et al. 1983) showed a remarkably close grouping of the four failure loads (2.95 to 3.03 times design load). This unusually close grouping could be indicating that a load-sharing effect may be at work, with all frame members being randomly paired and intimately interconnected at all joints.

The main purpose of this experiment was to evaluate two Canada Plan Service truss designs similar to frame geometry and member sizes, but having single versus double members. Where load-sharing applies, the Code for Engineering Design in Wood (Anonymous 1980) allows an increasing factor of 1.1 on wood working stresses for design. However, the code requires three (not two) parallel interconnected members in order to qualify for load-sharing. Our double trusses have only two parallel members in each part, but a degree of load-sharing is still possible because of the three-gusset, four-shear-plane nailed connection used. If a load-sharing effect exists, the double-member truss system should give failure loads more closely-grouped than the single-member system.

One further aspect of the load-sharing question is that the Canadian Farm Building Code (Anonymous 1983) allows the 1.1 load-sharing factor to be used for single-member trusses spaced at 1.2 m, but makes no mention of double-member trusses spaced at 2.4 m. Another consequence of the 2.4-m truss spacing is that the roof purlins (normally 38 × 89-mm members laid flat on trusses spaced at 1.2 m) must be fastened on edge (and sometimes doubled at support points) to handle the longer spans from truss to truss.

Truss lower chords may require special consideration. Because of the abrupt nature of a typical lower chord tension failure (and the possibility of zipper-type progressive building collapse as a result), the CSA Technical Committee on Engineering Design in Wood has recently proposed that the load-sharing factor for tension members be reduced from 1.1 to 1.0. This is a conservative decision from an engineering viewpoint. If adopted it will increase the size of some truss lower chords designed to meet the new code.

PROCEDURE

Five single-member trusses were assembled from CPS plan M-9136, 9.6 m span, and five double-member trusses of similar frame and gusset dimensions were made according to plan M-9220 (see Fig. 1 for both types). These trusses had been designed according to the CSA Code for Engineering Design in Wood (Anonymous 1980), with the allowable stresses used for determining the limiting design loads increased by a load-sharing factor of 1.5 and a low human occupancy factor of 1.25 as permitted by the Canadian Farm Building Code (Anonymous 1983). Thus the overall adjustment factor on allowable stresses was 1.1 × 1.15 × 1.25 = 1.58.

The distributions of design roof loads onto the panel points of the trusses were as shown in Figs. 2 and 3, taken directly from CPS plans M-9136 and M-9220, respectively. Both sets of trusses used outside gussets of 12.5-mm four-ply exterior sheathing Douglas fir plywood. In addition the double-member trusses used a single center gusset of 0.91 mm (20 gauge) galvanized sheet steel, sandwiched between the paired frame members and penetrated by 4 × 102-mm common spiral nails driven from both sides. Nails for the five single-member trusses were 4 × 64-mm concrete spiral nails (as specified in the CPS plans), also driven from both sides. Both joint systems had been previously evaluated (Turnbull et al. 1981), and the nail numbers at each joint were adjusted to give approximately equal joint deformations of 1.3 mm at the design load calculated for each truss type.

Lumber used for all 10 trusses was no. 2 (or better) S-P-F (spruce-pine-fir) species group, typical of the lumber available at eastern Canadian retail outlets. Lumber was probably all eastern or western spruce, although no thorough examination was made to check for other species. Both sets of trusses were made from the same shipment of lumber and plywood. Just before assembly the lumber was checked for moisture content using an electrical resistance type moisture meter. Moisture contents were generally over 28% (maximum scale for the meter) except for several of the upper chord 38 × 235-mm members (apparently shipped from older stock) which ranged from 16 to 23%. After assembly the trusses were space-stacked outdoors under a portable rain-roof. They were air-dried thus for 2 mo of mild late winter plus 1 mo of cool wet spring weather (−5° to +10°C). Final conditioning took place indoors in the testing laboratory, where the trusses were kept at about 18°C for an interim holding period of 2–3 wk.
Figure 1. Plan of single and double trusses. Numbers in squares are the numbers of nails driven from each side of each joint. Double trusses used 4 × 102-mm spiral common nails; single trusses used 4 × 64-mm spiral concrete nails.

Figure 2. Loading diagram and stress-ratio table for the single-member 9.6-m roof truss, from plan M-9136, based on trusses spaced at 1.2 m, design load 2.4 kN/m².

Figure 3. Loading diagram and stress-ratio table for the double-member 9.6-m roof truss, from plan M-9220, based on trusses spaced at 2.4 m, design load 2.4 kN/m².

When load-tested the lumber frames had air-dried to a mean of 16.3% moisture (range 11.8–21.1%). This procedure of assembling trusses ‘wet’ and testing them ‘dry’ was to simulate the real farm building situation. Typically, unseasoned lumber is nailed together at the farm construction site very soon after arrival from the lumber yard, but loading may occur months or years later when the trusses have had ample time to dry in service. With nailed truss joints, drying and shrinkage of the wood frames (without corresponding shrinkage of the nails) results in some spaces between the gussets and the frame members. This eliminates interface friction as an early source of joint stiffness, and is an important factor in modelling the ‘dry service’ condition assumed for a typical farm building attic.

Trusses were load-tested in the same order as they were built, to standardize the drying time as much as possible. Trusses were built, stacked and tested with single and double trusses alternating in sequence (to cancel any bias due to possible changes in the carpenters’ care or technique as the work progressed). Simulated snow loads were applied to the upper chords with a cable and pulley system at span intervals of 600 mm, as described previously (Turnbull et al. 1983). Loads were increased stepwise over a 2-h period, with 1/12 of full design load added every 10 min up to design load. Design load was held for 1 h, then the stepped loading was resumed and continued at the same rate up to failure.

Vertical deformations were recorded...
throughout the tests by three linear motion position transducers (potentiometer type), each located on the truss centerline and bearing on smooth steel plates screwed to the top of the three lower chord joints. The transducers were spring-loaded to 'follow' the deforming trusses to ensure that sudden truss failure would not damage the transducers.

Each truss failure mode was recorded, a moisture reading was taken in the wood members adjacent to each joint, then the trusses were dismembered with a chainsaw.

Under initial load, truss 1S (the first single-member truss to be tested) failed prematurely at the midspan lower chord joint. The load at this failure was below design expectations, apparently due to the use of four-ply plywood gussets. This was seen likely to cause subsequent failures and render the experiment inconclusive. Therefore truss 1S was repaired for re-testing, and all the single-member trusses including IS were reinforced with strips of 0.91-mm (20 gauge) galvanized sheet steel screwed in place about the weak lower chord joint. It was anticipated that this early failure indicated a need for an improved connection at this point and that the corresponding CPS plan would be quickly revised to reflect the needed improvement.

RESULTS AND DISCUSSION

Truss Deformations

Figure 4 shows deformation versus load curves for all 10 trusses. Loads were plotted in kN/m of span in order to discriminate between the single- and double-member trusses. During the 1-h hold period at full design load, the single trusses did not show any increased deformation (creep). However, four of the five double trusses (except truss 5D) each accumulated about 1 mm of creep deformation during the 1-h hold.

Table I (unlike Fig. 4) was based on equivalent roof loads expressed in kN/m² of horizontal roof area. This is a more equitable method of presenting the results, since we are really evaluating roof framing systems using single trusses at 1.2-m spacing on the same basis as double trusses at 2.4-m spacing. Table I summarizes the results of the loading tests. Column 3 gives the measured deformations at full design load. These deformations ranged from 19 to 23 mm for all trusses. Even the maximum deformation (23 mm) is only 1/417 of the span, which would easily satisfy most building code requirements for stiffness.

- **Table 1. Summary of Single-Member versus Double-Member Truss Performance**

<table>
<thead>
<tr>
<th>Truss plan</th>
<th>Truss no.</th>
<th>Deformation @ design load (mm)</th>
<th>Deformation @ failure (mm)</th>
<th>Load/design ratio</th>
<th>Roof load (kN/m²)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>M-9136, single-member, 9.6-m span</td>
<td>1S†</td>
<td>19.6</td>
<td>53</td>
<td>2.09</td>
<td>5.03</td>
<td>Lower chord gussets joint 2-2†</td>
</tr>
<tr>
<td></td>
<td>1S</td>
<td>23</td>
<td>94</td>
<td>2.67</td>
<td>6.42</td>
<td>Upper chord shear, around joint 4</td>
</tr>
<tr>
<td></td>
<td>2S</td>
<td>20</td>
<td>89</td>
<td>3.12</td>
<td>7.50</td>
<td>Heel gusset tension joint 1 (left)</td>
</tr>
<tr>
<td></td>
<td>3S</td>
<td>19</td>
<td>87</td>
<td>2.59</td>
<td>6.22</td>
<td>Heel gusset tension joint 1 (right)</td>
</tr>
<tr>
<td></td>
<td>4S</td>
<td>19</td>
<td>86</td>
<td>2.63</td>
<td>6.31</td>
<td>Lower chord tension span 1-2 (left)</td>
</tr>
<tr>
<td></td>
<td>5S</td>
<td>19</td>
<td>66</td>
<td>2.60</td>
<td>6.24</td>
<td>Heel gusset tension point 1 (left)</td>
</tr>
<tr>
<td>Means</td>
<td></td>
<td></td>
<td></td>
<td>20.0†</td>
<td>84.4†</td>
<td>2.72†</td>
</tr>
<tr>
<td>SD</td>
<td></td>
<td></td>
<td></td>
<td>± 1.73</td>
<td>± 10.74</td>
<td>± 0.22</td>
</tr>
</tbody>
</table>

†Truss 1S failed early; therefore, subsequent tests had the lower chord joints 2-2 reinforced with steel strapping to ensure failures would occur elsewhere. Mean values for the single-member trusses exclude the first test of truss 1S.

Numbers indicate failure locations referred to Fig. 2 and 3, loading diagrams.

- **Failure Loads and Failure Modes**

The failure load/design load ratios (Table I, column 5) can be used to assess the strength performance of the two truss groups. The single-member trusses failed at a mean of 2.72 times design load,
whereas the double-member trusses were significantly weaker ($P<0.05$), failing at a mean load of 2.29 times design load. This was not as anticipated, and requires some review of the observed modes of failure (column 7).

The single-member trusses with the reinforced lower chord midspan joints (2-2, Fig. 2) showed a random distribution of failure modes. These included one upper chord shear failure at the ridge joint (Fig. 5), one lower chord tension failure and three heel joint plywood tension failures (like Fig. 6). These failures demonstrate a design that could be further improved by upgrading the plywood gussets to the traditional five-ply material, but otherwise the design is well-balanced. The weakest of the single-member group (truss 3S) failed at 2.59 times design load, which is considerably above the 2.0 required by the Canadian Farm Building Code (Anonymous 1983). The break was a tension failure (Fig. 7) in the outspan lower chord member 1-2 (Fig. 2). Note in the stress-ratio table of Fig. 2 that this particular member is stressed at 0.78 times design, excluding secondary stresses due to truss deformation. As discussed previously (Turnbull et al. 1983), only the primary tension stress was considered in determining this lower chord stress ratio, thus allowing inadequate margin for the undetermined secondary stresses generated by truss deformations.

Three of the five double-member trusses failed in lower chord midspan gusset tension. Figure 8 is typical of this group. This was the joint corresponding to the reinforced joints in the other truss group. An extra depth of 50 mm had been added to the center steel gusset. This extra steel was bent horizontally under the bottom of the double-member joints, but this was obviously not sufficient reinforcement to eliminate this joint as the weak point in three of the five double-member trusses. On the other hand, the weakest double-member truss (5D, Fig. 9) did fail elsewhere (lower chord) at only 1.90 times design load. In this case, two paired members were weak in combined tension-bending within the same highly stressed lower chord span (1-2), although the weak points of the paired members were at knots at least 600 mm apart.

Figures 7, 8 and 9 also show some typical upper chord splits but these were all secondary; that is, they did not become obvious until after other (primary) failures had released the stored strain energy. Figure 5 shows one unforeseen upper chord splitting failure that occurred several times. This is primarily an upper chord compression joint that apparently suffers from a concentration of axial compression stresses at the bottom corners of the top chord members, due in part to upper chord-to-joint rotation. The observed splitting may be due to this stress concentration, resulting in shear failure, or it may be due to rebound when the truss breaks elsewhere. Approaching the point of failure it is too dangerous to closely watch the failure modes, and moreover it would be difficult to anticipate which joints should be watched.

The only evidence of any nail failure was seen in the heel joint of truss 3S (Fig. 10), where nail-heads pulled through one of the side gusset plates.
SUMMARY AND CONCLUSIONS
One objective of this experiment was to determine if doubling the frame members of trusses can give any load-sharing advantages such as more uniform safety factors or greater roof strength. Unfortunately for this purpose, failures in the joint gusset plates of both single-member and double-member truss groups effectively eliminated 7 of the 10 trusses tested. The remaining three trusses made an insufficient sample, and rendered this part of the experiment inconclusive.

At design load none of the trusses showed excessive deformation, the worst case being only 23 mm, or 1/417 of span (see Table I, truss IS). At the point of failure the double-member trusses deformed significantly less ($P<0.05$) than the single-member trusses, but this is probably because at failure the double-member trusses carried less load (mean 5.50 kN/m², versus 6.54 for the single-member group).

The most important conclusion from the experiment is that failures occurred mostly at the connections, not in the lumber frames. An obvious improvement is to replace the inferior 12.5-mm four-ply plywood with traditional five-ply plywood for outside gusset plates. Alternatively a designer could increase the size of the plywood gussets, but the lower chord tension gussets already suffer from some tension stress concentration at the bottom edge due to lower chord bending. Increasing the depth of these gussets would further increase the eccentricity of the axial loads on the plywood section and might not improve actual performance.

The double-member lower chord mid-span joints (as built) were strengthened over the original CPS design by adding more steel to the bottom edge of the center gusset, but some further improvement is still needed. A crude analysis of the mechanics of this type of nailed gusset connection (where each nail penetrates plywood-frame-steel-frame-plywood) shows that up to 5/8 of the total tension force probably is resisted by the single steel center gusset, and only 3/8 by the two plywood gussets. At the mean failure load of the five double-member trusses (5.50 kN/m² of roof), from member 2-2 (Fig. 3) the tension on the center steel gusset is estimated at (5.50/2.40) × 41.7 × 5/8 = 59.7 kN. Using the thickness and net effective depth of the steel gusset, this converts to a stress of 59.7/(0.91 × 184) = 0.356 kN/mm² = 356 MPa. This stress is well above the plastic limit for mild steel sheet (about 240 MPa), and shows that the center steel gusset is probably the limiting member in this critical connection. Increasing the steel thickness from 0.91 mm (20 gauge) to 1.22 mm (18 gauge) is one option to be further investigated.

Shear failures in the top chords about the ridge (joint 4, Figs. 2 and 3) have not received much design consideration. In the existing truss designs the ridge joint gussets were sized only for sufficient nailing area to transfer web tension into the upper chord. It may be advisable to increase the depth and nailing of the ridge joint gussets.
so that they cover almost the full depth of the upper chords.

With one exception (truss 5D), all trusses survived 2.0 times design load, although the trusses were not held at 2.0 times design load for 24 h as the Canadian Farm Building Code (1983) specifies.

Bearing in mind that the specified design loads for all of these trusses were increased by the combined effect of load duration, load sharing and low human occupancy factors \((1.15 \times 1.10 \times 1.25 = 1.58)\), it would appear that adequate safety can be achieved by modifying the weak points in each design and continuing to use all three factors. Inexpensive design improvements such as relocated gusset plates and member splices are obviously more economical than de-rating the truss design loads.

ACKNOWLEDGMENTS

The authors acknowledge the contributions to this work by J.-M. Leclerc (technologist), engineering students Kevin McKague and Scott Laking who completed the testing, and carpenters A. Wiskowski and R. McAdam who assembled the trusses.

REFERENCES


