Design method for 1986 Canada Plan Service roof trusses

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Massé, D. I., Turnbull, J. E. and Jackson, H. A. 1988. **Design method for 1986 Canada Plan Service roof trusses**. Can. Agric. Eng. 30: 277–281. The new approach used by the Canada Plan Service to optimize the roof truss design is given in detail. This paper describes the analog model used to represent more closely the real structure and covers the design procedure for sizing the plywood gussets and nailing patterns to resist shear, tension and flexural forces developed in the joints.

INTRODUCTION

Previous Canada Plan Service (CPS) trusses were designed with most joints considered as pinned. The connections were previously made with 12.5-mm, 5-ply Douglas fir plywood and 63-mm concrete nails. The 5-ply plywood previously used has virtually disappeared from the Canadian market, and has been replaced by 4-ply, with only the two face veneers running parallel to the panel length. This new plywood is not satisfactory for tension gussets, therefore a revised connection system using thicker 18.5-mm plywood and larger 76-mm concrete nails is described.

This report summarizes a new computer-aided design procedure based on a stiffness method of analysis that is used to optimize the design of CPS roof trusses. Part I of this paper describes a frame analysis which closely represents the real structure. Part II covers the design procedure used to size the gussets to resist shear, tension and flexural forces developed in the joints. This includes specification of the nailing details based on nail lateral strength resistance taken from experimental work (Massé et al. 1986), and based on the minimum nail spacing requirements as tested.

PART I DESIGN METHOD FOR TRUSS MEMBERS

The new design method takes advantage of an advanced computer programme. 'Purdue Plane Structures Analyzer II'' (PPSA II) (Suddarth and Wolfe 1984) was modified to Canadian code requirements and SI units. With this programme the analog model very closely represents the real structure, and the stress distribution and truss deformations are accurately predicted. At design load, predicted truss deformations were within 12% of average measured deformations for 5 trusses (Massé 1985).

Figure 1 shows a typical CPS single-W gable roof truss. The heel and lower chord joints are both large and fastened with many nails. Such joints cannot be considered as pinned. In a previous study (Massé 1985) the rotational stiffnesses of these joints were found to be at least as great as the bending stiffness of the corresponding connected frame members. Therefore, in the analog model these joints are considered as rotationally rigid (Fig. 2). On the other hand, the web connections and ridge joint, having relatively few nails, can develop only a small resisting moment. For these joints it is realistic to assume a pinned connection (Fig. 2).

The upper chord splice is located as closely as possible to a point of zero moment. This way the upper chord can be considered as a single, continuous member in the analog model.

All joints in the truss are considered free to deform axially, even the joints that connect compression members. A visual inspection of trusses tested in the Engineering and Statistical Research Centre (ESRC) laboratory showed that tight butt joints are seldom achieved, due to wood shrinkage and workmanship error. Therefore the analog model is given fictitious axial springs, shown in series with each hinge (Fig. 2), to allow for tension and compression deformations. The methodology to determine the stiffness of the fictitious springs is given by Massé (1985).

For most joints the member centerlines pass through a single point. However, at the heel joint, the vertical support reaction does not pass through the intersection of the centerlines of the upper and lower chords. The eccentricity at that location is not negligible, therefore a short vertical fictitious member having a very large stiffness and a length equal to that eccentricity is inserted at the heel joint (Fig. 2).

The roof dead and live loads are represented in the analog model by uniformly distributed vertical load applied along the upper chord.

Using this model, CPS Truss plan M-9143 (Fig. 1) is analyzed for load case I (0.2 kPa uniform dead + 2.05 kPa uniform snow) and load case II (0.2 kPa uniform dead + 2.5 kPa unbalanced snow). Other assumptions are No. 2 S-P-F lumber, truss spacing 1.2 m, low human occupancy farm building and dry service. For loading case I (Dead + Uniform snow load) the moment distributions along the upper and lower chords are shown in Fig. 3 and the axial force, bending moment and combined stress ratio for each truss member are given in Table I. In the case of plan M-9143 the maximum allowable combined stress ratio in the upper chord inner span is 1.00, based on the wood code (Technical Committee on Engineering Design in Wood 1984).

Turnbull and Massé (1987) commented on results from 3 yr of full-scale truss testing. Of 25 trusses tested, only two failures occurred in top chords. One of these was a shear failure, and the other was a bending failure in a region of zero axial compression. Almost all truss failures occurred in the lower chord members at their tensile connections. Buchanan et al. (1985) have proposed a possible explanation for the apparent overstrength of the truss upper chords by showing that the code

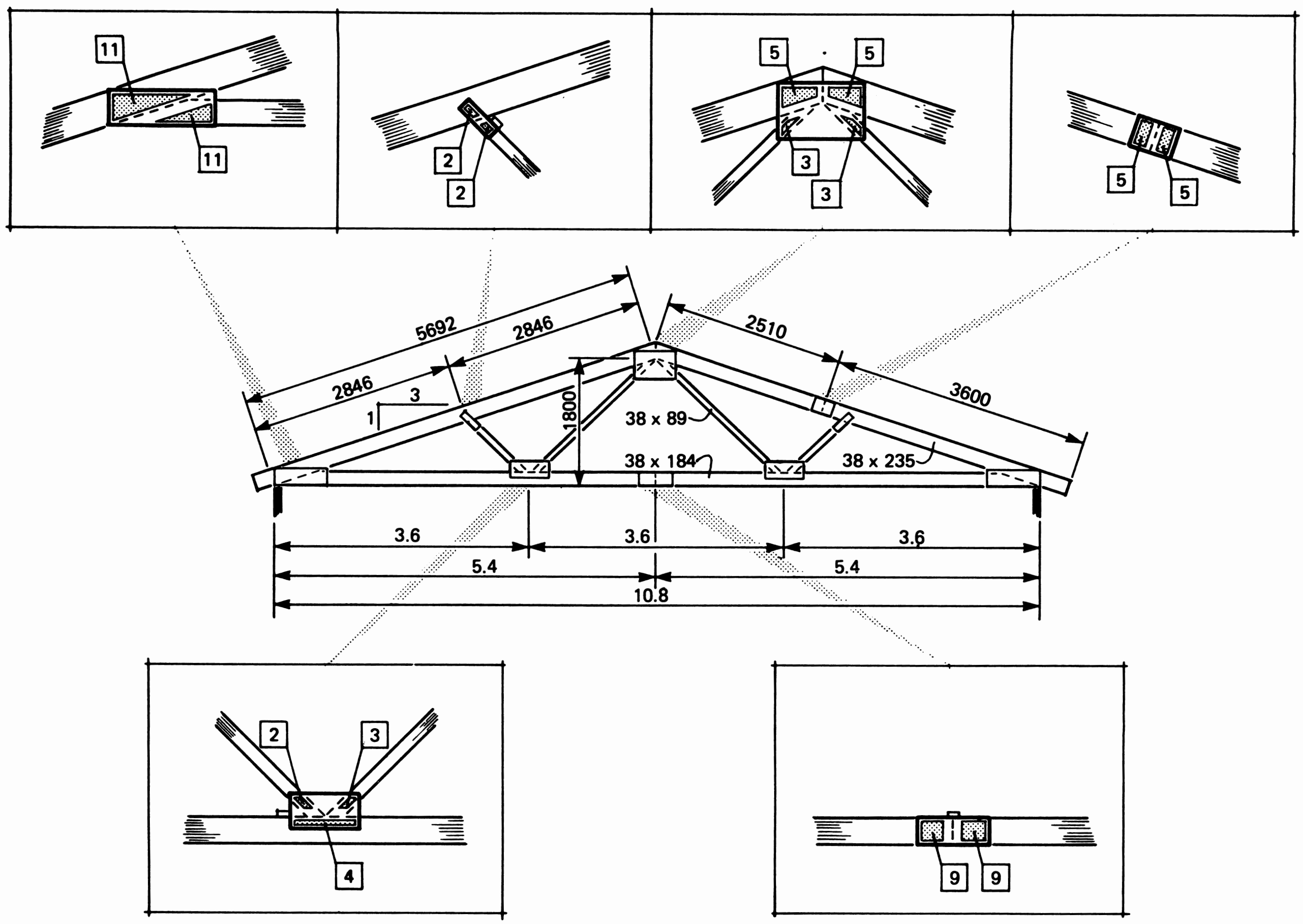

Figure 1. CPS Plan M-9143 – Typical single W, double slope CPS truss.

is too conservative concerning the design of members under combined bending and axial compression.

Figure 4 is adapted from Buchanan et al. (1985). The interaction curves at the lower fifth, mean and 95th percentile show that the combined bending moment and axial compression capacity of short columns increase when an axial compressive force of a specific range is superimposed on bending. Supplementary tests done on deeper members showed the same tendency. Also shown in Fig. 4 are the code interaction curves, one straight line for bending plus compression, another for bending plus tension. It can be seen that the code interpretation is very conservative for some combinations of axial compression force and bending moment when compared to the lower fifth percentile interaction curve.

Buchanan's proposal is now before the code committee and may be accepted for the next edition. In the interim, the authors at the CPS Design Center have chosen to hold the combined stress ratio at ≤ 1.0 until the code committee has decided on appropriate action.

The 1986 design roof load for CPS truss plan M-9143 is shown in Table III, giving a balanced roof load 12% higher than previously. Part of this increase is due to the stronger gusset connections and part is due to the computer-aided truss analysis method that gives a better estimate of critical bending moments in the upper and lower chords. Other trusses in the 1986 series have greater or lesser increases. These designs are better balanced than previous versions; that is to say, the failure locations are more likely to be randomly distributed throughout the truss members and connections.

The uniform design load in Table III is governed by the critical frame members (1) and (4) (Fig. 2), where the combined stress ratios are both 1.00. For the unbalanced (case II) loading, shear (rather than combined bending and compression) is found to control the design of the truss upper chords.

PART II DESIGN PROCEDURE FOR NAILED GUSSET JOINTS

The size of a plywood gusset is a function of the strength capacities in tension and shear through its thickness. Code strengths (Technical Committee on Engineering Design in Wood 1984) for 18.5-mm 5-ply Douglas fir plywood were used to determine the gusset length and depth. Figure 5 shows a typical heel joint. Fracture line (1) shows the critical failure path (shear failure). Therefore the minimum gusset length to resist shear according to clause 7.5.5.2 of the Code (Technical Committee on Engineering Design in Wood 1984) must be at least

$$L = P/91 \qquad (1)$$

where:

L = gusset length (mm);
P = lower chord exterior span force (N);
91 = factored shear strength of two gussets (N/mm of length).

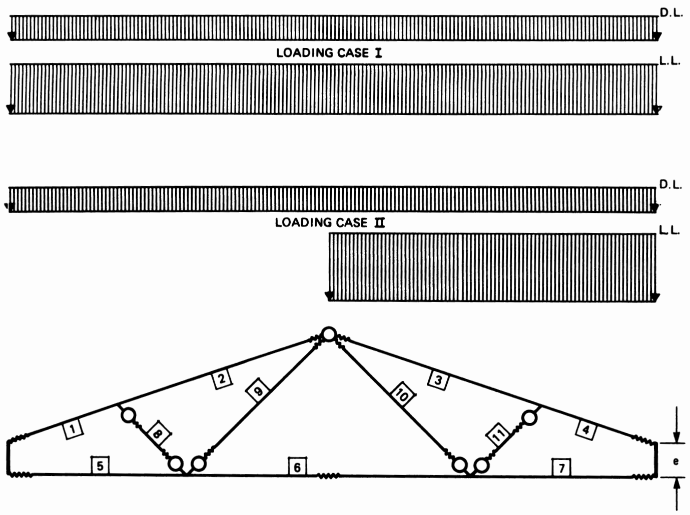

Figure 2. Analog model for CPS single W, double slope trusses.

The length is also checked in order to respect the minimum nail spacing requirements.

Fracture line (2) (Fig. 5) shows the assumed critical path for tension. Gusset depth is controlled by the combined joint axial force and bending moment. Combining equations from code clauses 7.5.3. and 7.5.7 (Technical Committee on Engineering Design in Wood 1984) the depth is calculated as follows:

$$D = \frac{0.00345\,P \pm \sqrt{(0.00345P)^2 + (0.083\,M)}}{2} \qquad (2)$$

where:

D = gusset depth (mm);
P = axial force acting on the joint (N);
M = bending moment acting on the joint (N.mm).

The lower chord splice length is controlled by the nail spacing requirements but the depth is controlled by the combined axial force and bending moment (Eq. 2).

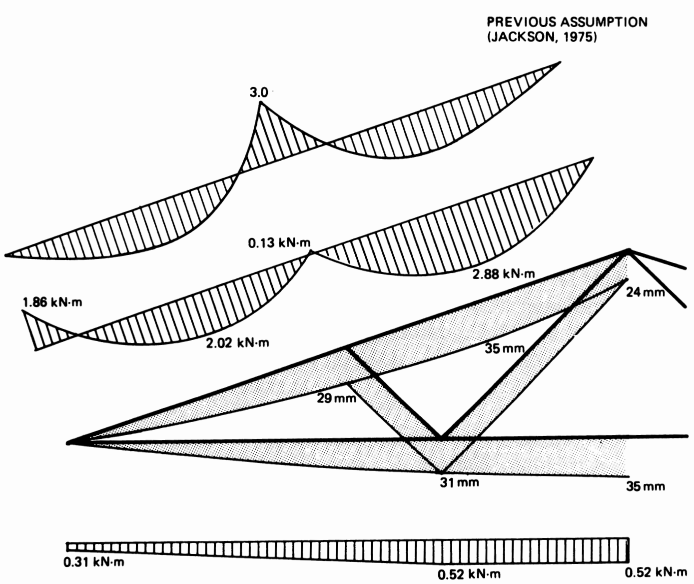

Figure 3. Vertical deformations and moment distributions along the truss upper and lower chords.

Table I. Factored forces, moments and stress ratios (dead load + uniform snow) in truss members, plan M-9143

Member	Axial force (P) (kN)	Maximum moment (M) (10^6N.mm)	Stress ratio† $P_u/AF_i + M/SF_b$
1	−40.8	2.03	0.90
2	−34.5	2.89	1.00
3	−34.5	2.89	1.00
4	−40.8	2.03	0.90
5	37.3	0.52	0.86
6	25.4	0.52	0.66
7	37.3	0.52	0.86
8	−8.5	0	0.53
9	8.5	0	0.31
10	8.5	0	0.31
11	−8.5	0	0.53

†P_u = factored force; M = factored moment; A = area; S = section modulus; F_i = specified strength in compression or tension; F_b = specified strength in bending.

Table II. Factored forces and stress ratios (dead load + unbalanced snow) in truss members, plan M-9143

Member	Axial force (P) (kN)	Maximum moment (M) (10^6N.mm)	Stress ratio† $P_u/AF_i + M/SF_b$
1	−17.4	0.58	0.33
2	−17.1	0.60	0.33
3	−26.2	3.13	0.94‡
4	−33.9	2.31	0.87
5	16.2	0.1	0.33
6	15.9	0.76	0.53
7	30.7	0.76	0.81
8	−0.0	0	0.00
9	0.0	0	0.00
10	10.4	0	0.38
11	−10.4	0	0.66

†P_u = factored force; M = factored moment; A = area; S = section modulus; F_i = specified strength in compression or tension; F_b = specified strength in bending.
‡For nonuniform snow load, shear in top chord member 3 controls the design.

Table III. Total permissible roof snow + dead load (kPa) for different truss spacings, Plan M-9143

	Truss spacing (mm)		
	600	800	1200
Uniform	4.3+0.2	3.2+0.2	2.1+0.2
Unbalanced	5.1+0.2	3.8+0.2	2.5+0.2

The gussets for the web-to-chord connections, the upper chord splices and the ridge joint are sized to meet nail spacing requirements, resulting in overdesign in all other respects.

Concrete nails, 4.5×76 mm, were chosen to completely penetrate all three members in the connections. With working stress design (WSD), the nail allowable lateral capacity as determined by test (Massé et al. 1986) was 1.25 kN at a joint displacement of 1.27 mm. To use the Limit States Design (LSD) method it was necessary to calibrate from the nail allowable lateral capacity to obtain the nail lateral strength resistance.

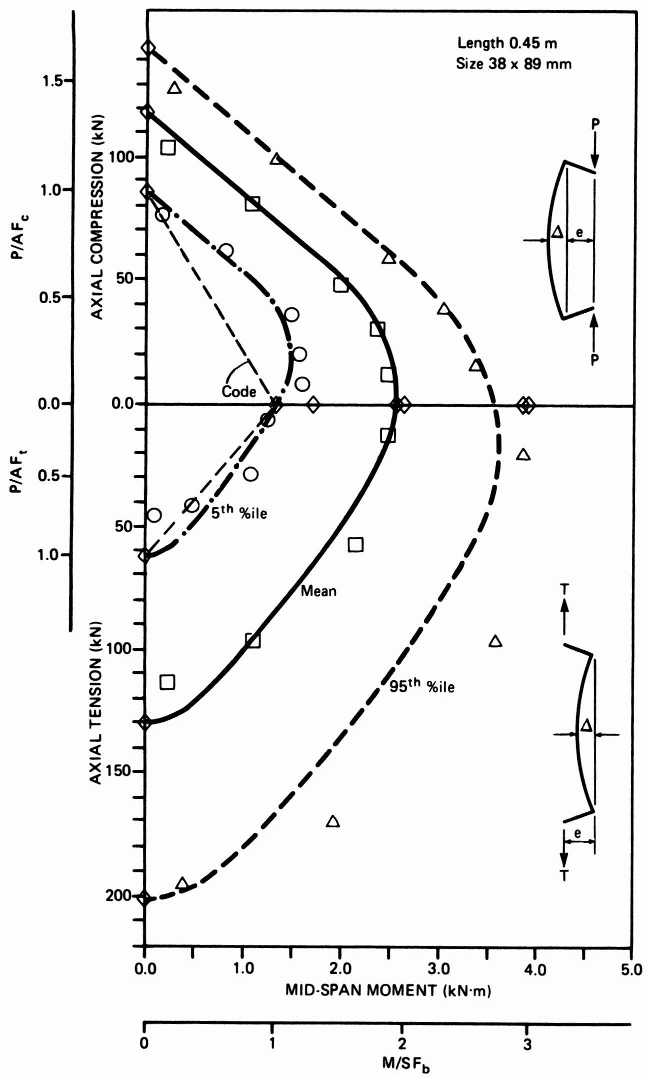

Figure 4. Strength of sawn lumber members subjected to various combinations of bending and axial stresses (adapted from Buchanan et al. 1984).

This was done by assuming that both WSD and LSD would require the same number of nails per joint. In the WSD method

$$\text{No. of nails} = \frac{P}{N_{u_{WSD}} K_i} \tag{3}$$

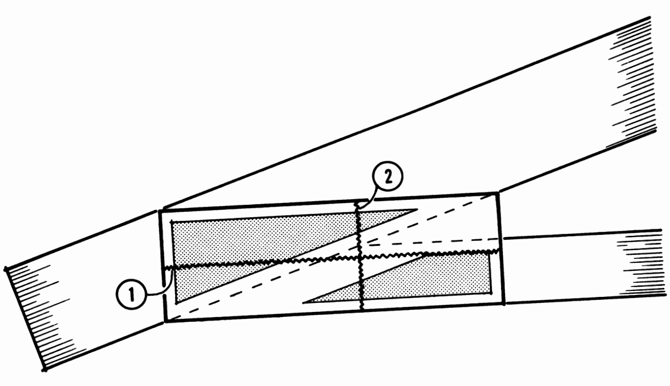

Figure 5. Critical failure paths for heel joint gussets.

where:

P = force acting on the joint (N);
$N_{u_{WSD}}$ = allowable double shear nail lateral capacity (N/nail);
K_i = product of modification factors.

In the LSD method

$$\text{No. of nails} = \frac{P_u}{\text{Ø} N_{u_{LSD}} K_i} \tag{4}$$

where:
P_u = factored force = 1.5 P (N);
Ø = resistance factor = 0.7 (code);
$N_{u_{LSD}}$ = double shear lateral strength resistance (N/nail);
K_i = product of modification factors.

From Equations 3 and 4

$N_{u_{LSD}} = 1.714 N_{u_{WSD}} = 2.1$ kN/nail

With respect to the lumber members, the nail spacings specified on CPS plan M-9143 are identical to those used in the test joints (Massé et al 1986), as follows:

Loaded end distance: 75 mm
Edge distance: 25 mm
Nail spacing parallel to grain: 50 mm
Nail spacing perpendicular to grain: 30 mm

Except as noted below, the above nail spacings were based on the authors' experience with plywood side-plates and large nails in spruce lumber, being staggered and spaced considerably closer than the minimum spacings tabled in the code (Technical Committee on Engineering Design in Wood 1984). The exception was lumber loaded-end distance, which was increased from code (15 diameters × 4.5 mm = 67.5 mm).

SUMMARY AND CONCLUSIONS

The CPS Roof truss plan M-9143 was revised using new joint details and a more advanced design method. For the truss used for illustration in this paper, the allowable roof load is increased by 12%. Truss connections made with thicker plywood gussets (18.5-mm, 5-ply Douglas fir exterior sheathing) and bigger, longer concrete nails (4.5 × 76 mm) required 25% fewer nails and less plywood area than previous truss plan M-9143. It is considered that the overall design is now better-balanced with more efficient use of all truss materials.

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