BRITISH COLUMBIA Ministry of Agriculture, Food and Fisheries

Agricultural Building Systems Handbook

PLAN 305-12

STEEL ROOF DIAPHRAGM WIND BRACING WITH STUD WALLS

COMPLETE INSTRUCTIONS

DEVELOPED BY CANADA PLAN SERVICE

STEEL ROOF DIAPHRAGM WIND BRACING WITH STUD WALLS

CPS
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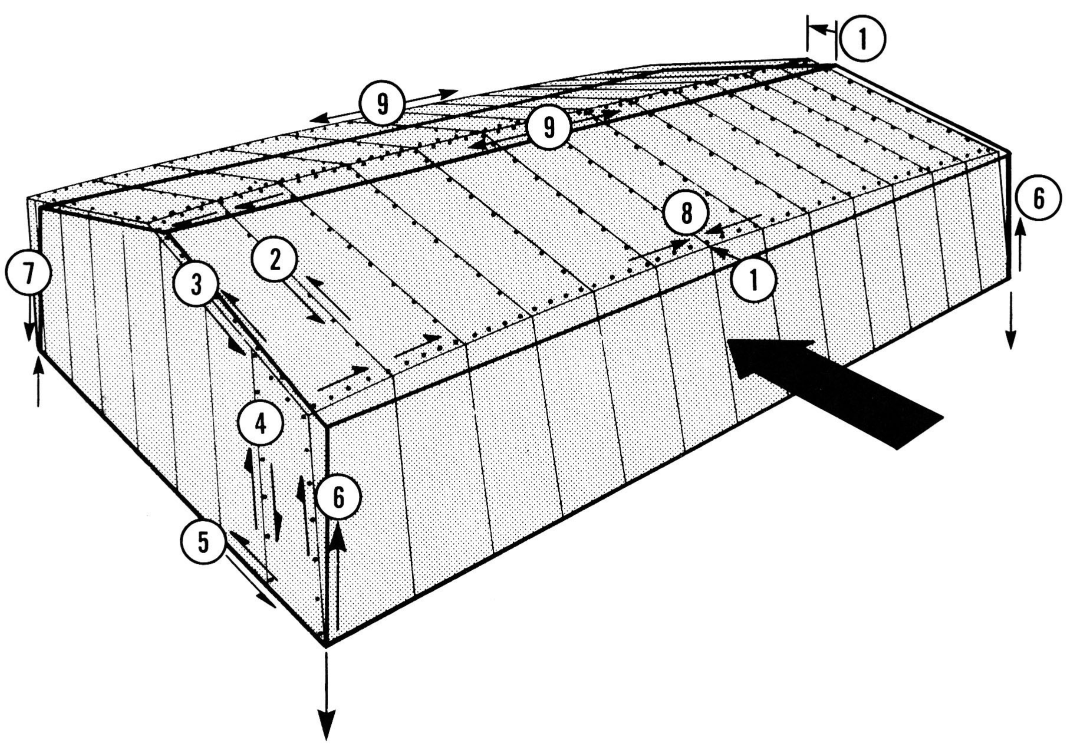
This plan is for stud-wall farm buildings having almost no resistance to overturning at the stud-to-foundation connection. These specifications will be over-safe if used with pole-frame walls because the poles add some overturning resistance. The tables and charts included here will be equally applicable to a diaphragm roof used to brace the top story of a two- or three-story stud wall farm building.

Wind blowing across a typical gable-roof farm building produces forces acting perpendicular to the surfaces. Wind tries to lift the roof and overturn the walls. The uplift forces on roofs can be, handled by secure attachment of roofing to trusses, trusses to walls, and walls to foundations; the horizontal overturning forces must be handled in other ways.

Knee-bracing from truss-to-wall is one way to resist these horizontal forces. Another way is to use pole-frame wall construction, with oversized poles to handle wind. A steel roof diaphragm is a third method, and since many farm buildings have a steel roof anyway, this is often the least expensive.

For effective diaphragm action, each panel of roof and wall cladding must be connected along all four edges to adjacent framing and cladding (Figure 1). This makes the entire building work like a rigid box to resist wind forces. The plan gives details of the extra cladding and framing connections necessary to make an effective roof/endwall diaphragm wind-bracing system.

WORKING STRENGTH OF ROOFING STEEL AND ROOFING SCREWS In Canada the customary steel roofing thickness for wood-framed farm buildings is 0.3 mm (30 gauge, before galvanizing), with ribs spaced at 150 to 200 mm. The roofing shear strength may be limited by either the buckling shear resistance of the ribbed steel sheet, or by the shear resistance of the roofing screws driven around the perimeter of each sheet. Roofing profiles with ribs screwed down to the purlins at 150 mm spacing are safe to about 3.7 kN/m of sheet length or width. However, some new high-rib



1 sidesway due to wind is exaggerated to illustrate how the diaphragm roof works
2 stitch-screws at the roofing edge-laps carry the wind shear forces to the ends of the roof
3 roofing screws and blocking carry the forces into the gable truss
4 endwall cladding carries the forces from the gable trusses down the endwalls
5 cladding-to-foundation shear connection
6 uplift forces at windward foundation corners
7 leeward corner forces may be up or down, and are less critical than ⑥
8 roof bending causes compression at the windward edges, and
9 tension at the leeward edges of the roof planes

Figure 1 A steel roof diaphragm carries the overturning wind force into the endwalls and foundation

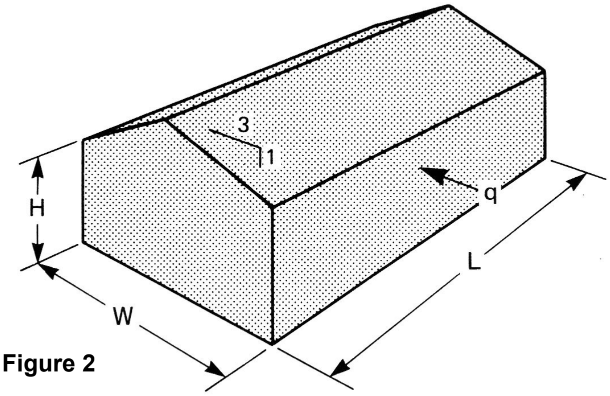


Figure 2

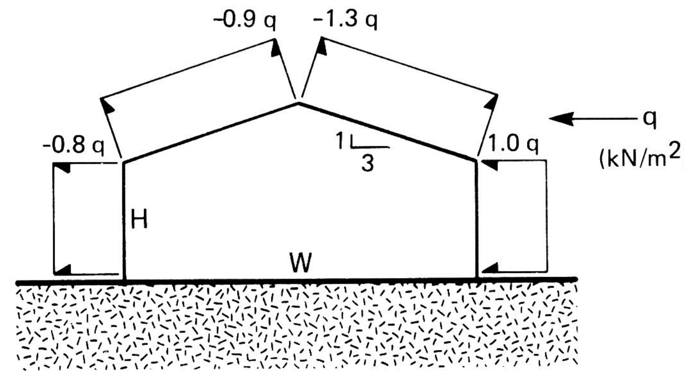


Figure 3

roofing profiles are ribbed at a wider spacing, about 300 mm. Use these profiles for diaphragm roofs only if a 300 mm screw spacing is safe at the side-laps and around the roof perimeter. This is to make sure that these wide-rib profiles are not as highly stressed.

Research gives a safe shear of 0.45 kN for a typical 4 x 25 mm (No. 8 x 1 in.) hex-head roofing screw when used as a stitch-screw (driven through the lapped edges of two sheets of 30-gauge steel roofing). Screws hold better than nails, and they can be used as side lap stitch-screws even where there is no wood framing under the steel.

Spacing of the perimeter screws along the ends of the roofing sheets is controlled by the rib spacing, and along the side-laps by the purlin spacing (side-laps may be shear-reinforced by driving one to three short stitch-screws equally spaced between each pair of roof purlins). A stitch-screw spacing of 150 mm makes a lap joint that has shear strength at least equal to that of the 30-gauge steel.

DESIGN Wind pressures for locations across Canada, and the rules for determining design wind forces on various shapes of buildings, are tabled in the *Supplement to the National Building Code of Canada, 1985*. For low human occupancy farm buildings, the Canadian Farm Building Code 1983 permits the use of the 1/10 hourly wind pressures, as tabled in the *Supplement.*

For rectangular farm buildings with stud walls and gable truss roofs (Figures 2 and 3), the total horizontal wind force on windward and leeward walls per metre of building length is approximately (1.0 + 0.8) qH, which includes the 2.0 gust factor in the pressure coefficients 1.0 and - 0.8 (a change in the 1980 code, as compared to previous codes). The roof-pressure coefficients - 1.3 and - 0.9 (Figure 3) also have horizontal components, but they almost cancel and are neglected here.

Since with stud walls about half of the total overturning wind force goes directly to the long wall foundations, the total shear force (V) where the two roof planes meet the gable end truss is

$$V = (1.0 + 0.8)\ qHL/4$$

and the stitch-screw spacing S to carry this shear force is

$$S = 0.45 \frac{W}{V} = \frac{(0.45)\ (4)}{(1.0+0.8)} \frac{W}{qHL}, \text{ therefore}$$

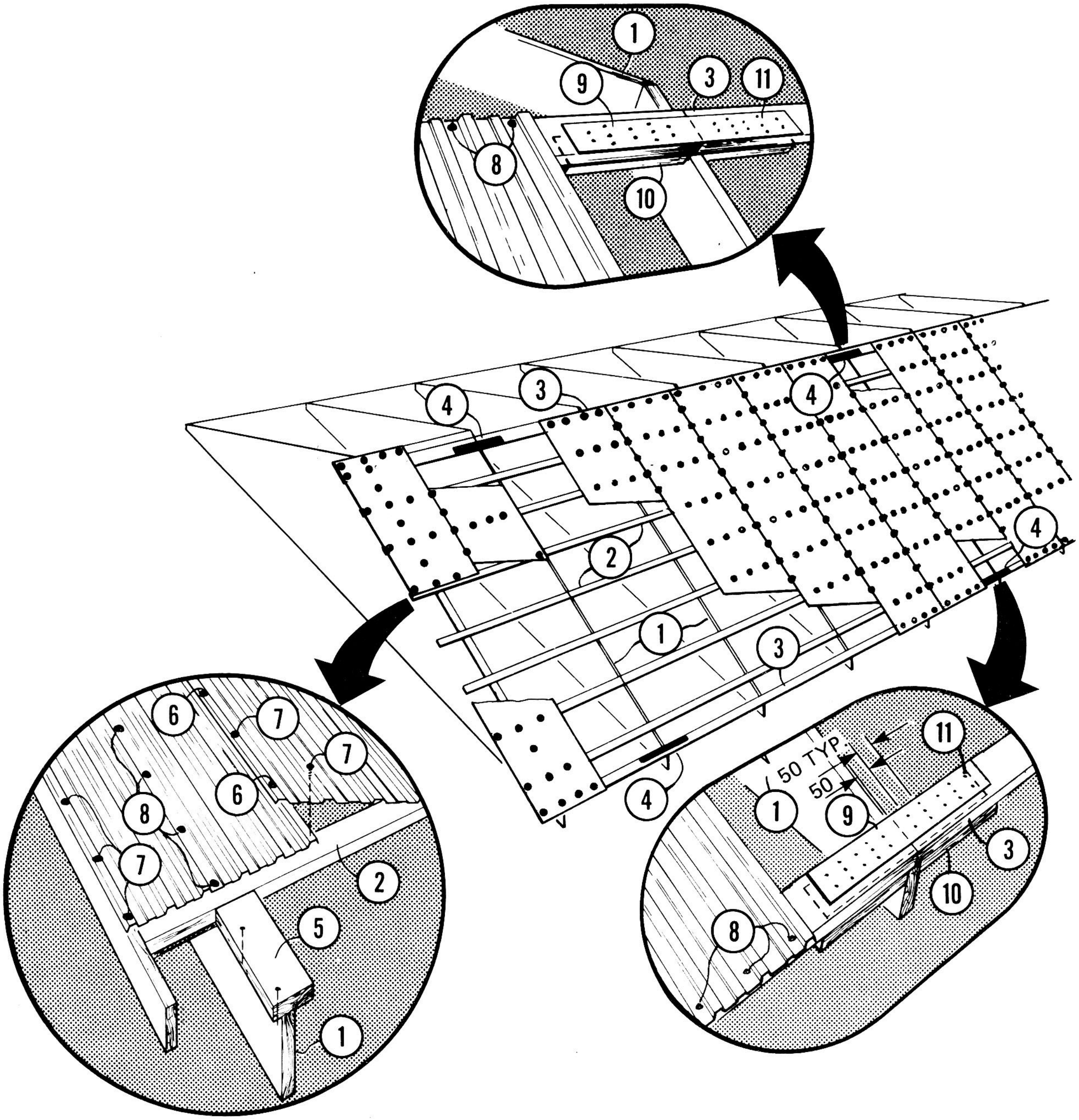
$$S = 1.0 \frac{W}{qHL} \text{ metres.}$$

Solve the above equation for S in each case, or enter the appropriate building and wind force dimensions into Figures 6, 8, 10, 12, or 14.

DESIGN FOR BENDING Bending moment is the other item to consider when designing the roof diaphragm. The two roof planes can be considered as flat beams loosely connected edge-to-edge at the ridge by the roof trusses. This plan gives design information for splicing the roof edge purlins (at eaves and ridge) into continuous tension and compression members to resist bending in the two roof diaphragms.

Figure 4 shows the location and details for each type of connection. The idea is to make very strong tension splices in the 38 x 140 mm edge purlins, at 4.8 m length intervals. The splice is made by two strips of 0.91 mm (20 gauge) galvanized steel to sandwich the purlin ends. These are nailed through with 4.5 x 75 mm (3-in.) concrete nails. These hardened nails are capable of penetrating both steel strips without predrilling, although some builders may prefer to predrill undersized nail holes, especially to help the nails penetrate the top strip. Figure 4 also shows short 38 mm backing blocks ⑩. These are held tightly under each half-joint with C-clamps while driving the concrete nails. The blocks remain to support the bottom steel strip and hold the nail points (easier and more effective than clinching). Each nail is estimated to hold 2.0 kN (after adjusting for wind load duration, two shear planes, steel-to-wood, low human occupancy, etc.).

The bending assumptions are shown in Figure 5. The maximum moment (M) occurs at mid-length and would be $M = 0.1125\ qHL^2$. Assuming that the entire moment



1 endwall roof truss

2 38 x 89 x 4800 mm roof purlins @ 600 mm oc, ends staggered 2400 mm

3 38 x 140 x 4800 mm edge purlins, spliced continuous

4 splice detail for ③

5 38 x 140 mm blocking between ② at ①; 89 mm spiral nails to end truss ①at same spacing (S) as screws⑥

6 #8 x 25 mm stitch-screws @ lapped edges of roofing, spaced according to Figure 6, 8, 10, 12 or 14

7 #8 x 38 mm roofing screws to each purlin at ribs

8 #8 x 25 mm roofing screws (on flats at roofing) to ⑤ same spacing as screws ⑥

9 0.91 (20 ga.) x 100 mm galv. steel straps, 2 per joint in ③, length to suit number of nails (N) per half-joint (see table)

10 38 x 140 mm blocking both sides of truss @ ④

11 4.5 x 76 mm special concrete nails thru ③, ⑨ & ⑩ allow 2.0 kN/nail; N nails each half of joint (Figure 7, 9, 11, 13 or 15)

Figure 4 Diaphragm steel roof with special connections at roofing side-laps, roofing to endwalls, roofing to edge purlins, and edge purlin splices

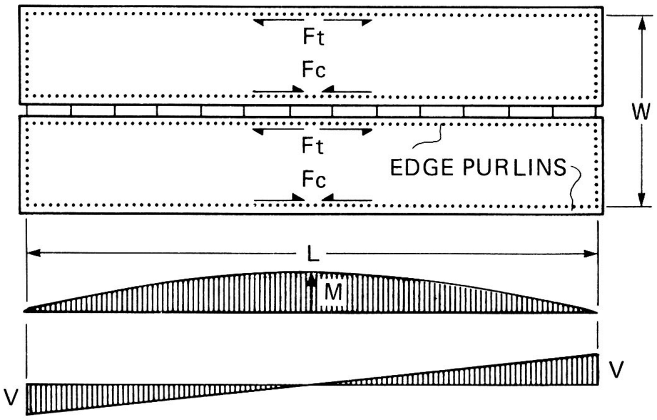


Figure 5 Roof plan with bending moment and shear diagrams

is resisted by tension and compression in the four edge purlins and that

$F_t = F_c$, then

$M = 0.1125\ qHL^2 = 2F_t\ (W/2)$, from which

$$F_t = 0.1125\ qH\frac{L^2}{W}$$

The number of 75 mm concrete nails in each edge purlin half-joint (Figure 4) is therefore $F_t/2.0$, and the nail number (N) per half-joint is

$$N = \frac{0.1125}{2.0}\ qH\frac{L^2}{W} = 0.056\ qH\frac{L^2}{W}$$

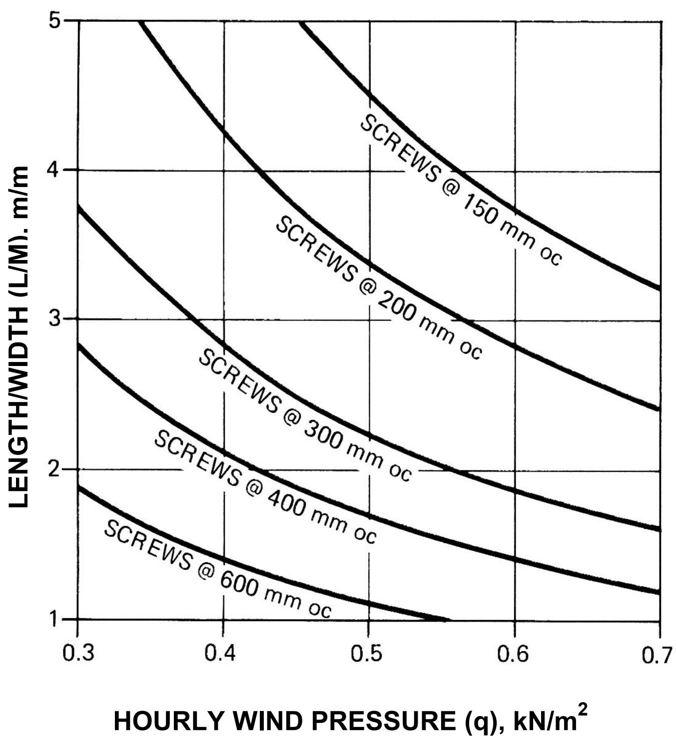


Figure 6 Roofing stitch-screw spacing, wall height H = 2.4 m

For graphic solutions to nail number N, see Figure 7, 9, 11, 13 or 15. Calculate L^2/W (in metres) before entering the graphs. Note that the safe tensile strength of 100 mm wide steel straps will be exceeded at over 12 nails (see*). Above this line, increase the straps to 140 mm wide. Use the following table to find the steel strap length required.

	Strap length (mm) for nail number (N) of						
	2	5	7	10	12	15	20
Straps ⑨ 0.91 x 100mm	200	300	400	500	600		
Straps ⑨ 0.91 x 140 mm						600	700

If the two roof planes can be connected to effectively transfer shear across the ridge, you can omit the ridge purlin connections shown at Figure 4. This makes the roof diaphragm considerably stiffer; however it is often not practical because of roof-ridge ventilators, etc.

ENDWALLS The diaphragm roof is designed to carry the horizontal wind force into the endwalls; there they must act as shear-walls at least as strong as the roof. Figure 4 shows how to fasten both ends of the roof diaphragm to the endwall truss and framing. Screws ⑧ should be driven on the flats of the roofing, at the same spacing as the edge lap stitch-screws (Figures 6, 8, 10, 12 or 14).

*for 15 nails or more, increase straps to 140 mm wide

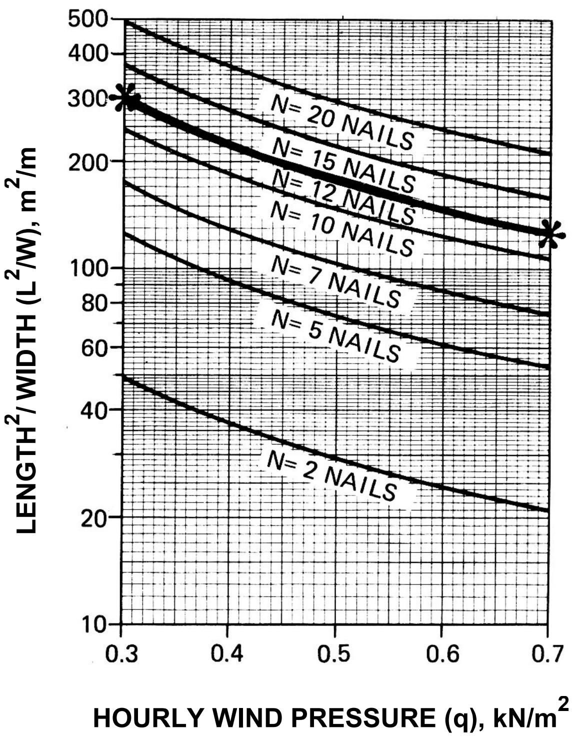


Figure 7 Edge-purlin splice nailing, wall height H = 2.4

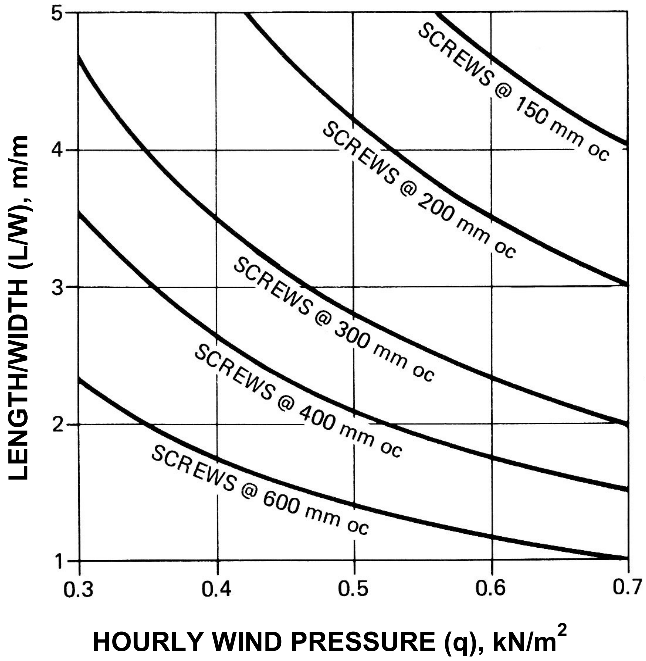


Figure 8 Roofing stitch-screw spacing, wall height H = 3.0 m

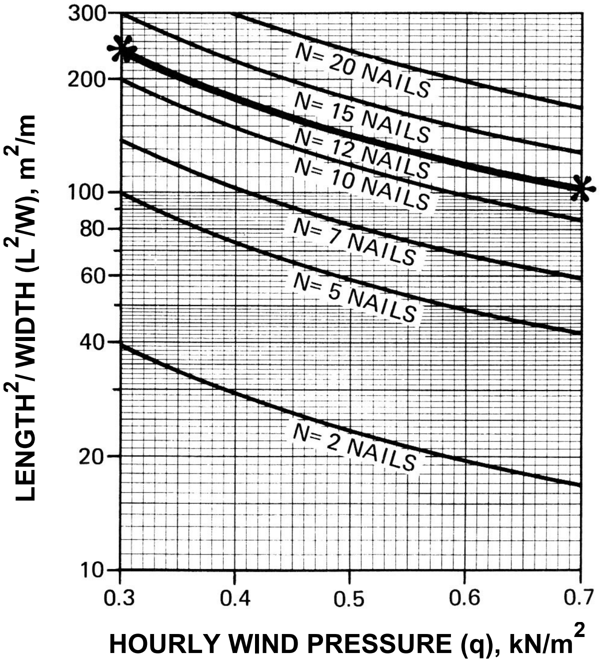


Figure 9 Edge-purlin splice nailing, wall height H = 3.0 m

*for 15 nails or more, increase straps to 140 mm wide

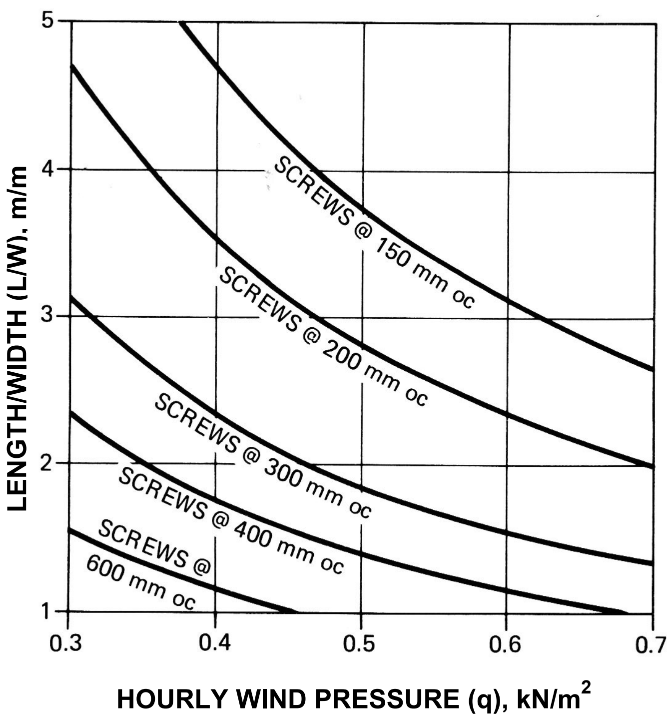


Figure 10 Roofing stitch-screw spacing, wall height H = 3.6 m

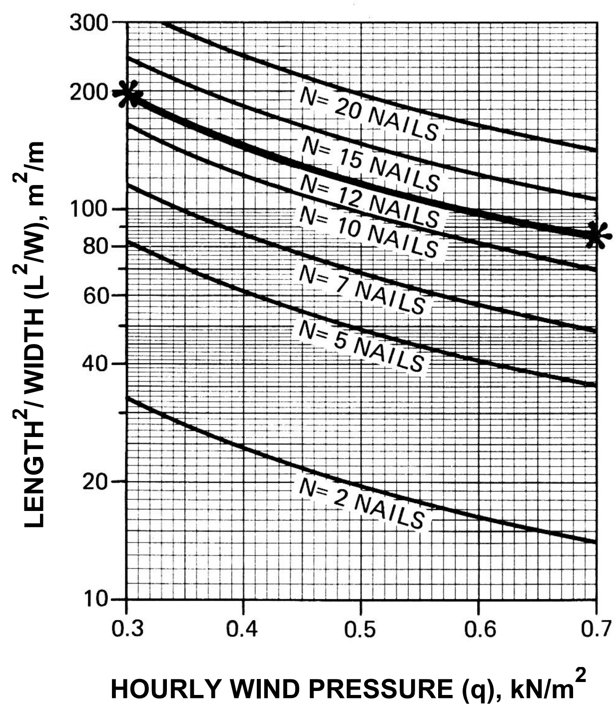


Figure 11 Edge-purlin splice nailing, wall height H = 3.6 m

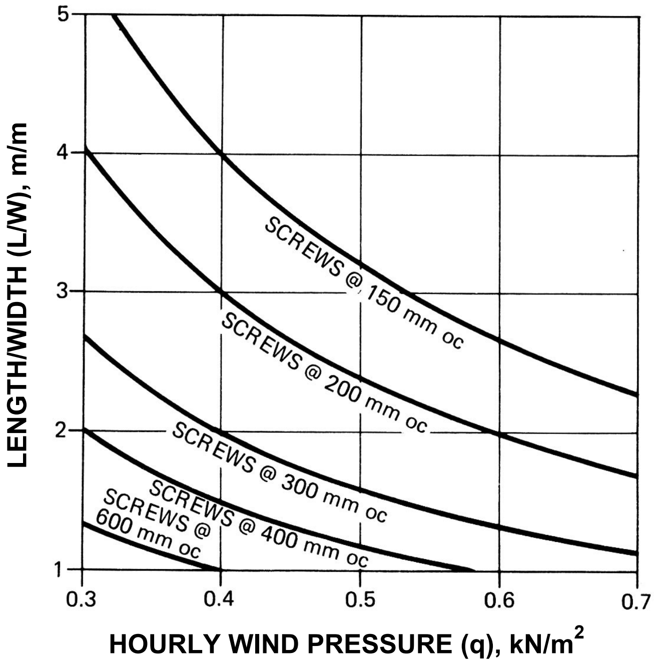


Figure 12 Roofing stitch-screw spacing, wall height H = 4.2 m

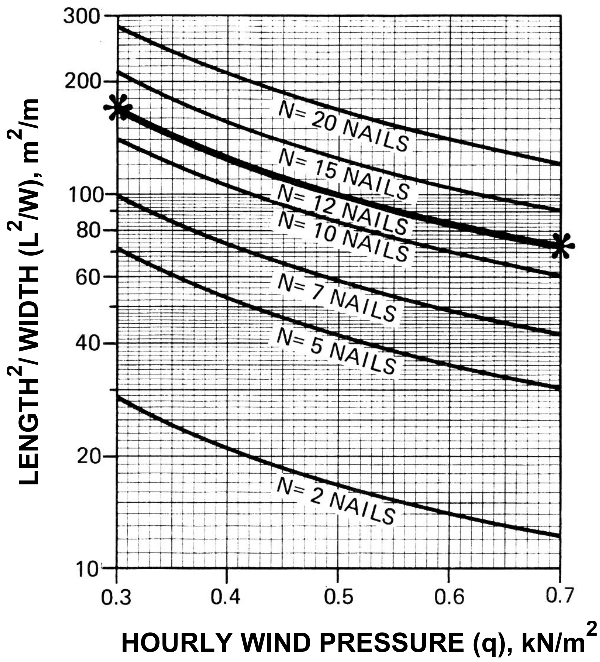


Figure 13 Edge-purlin splice nailing, wall height H = 4.2 m

*for 15 nails or more, increase straps to 140 mm wide

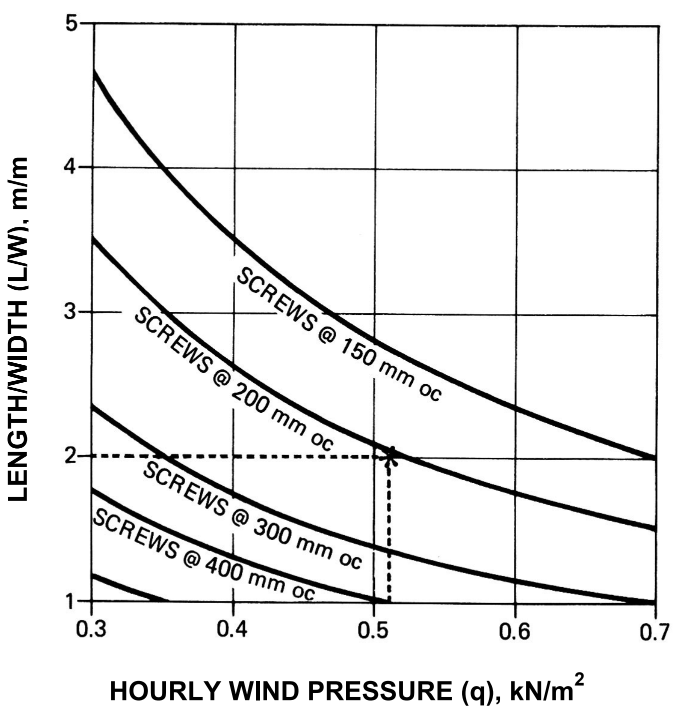


Figure 14 Roofing stitch-screw spacing, wall height H = 4.8 m

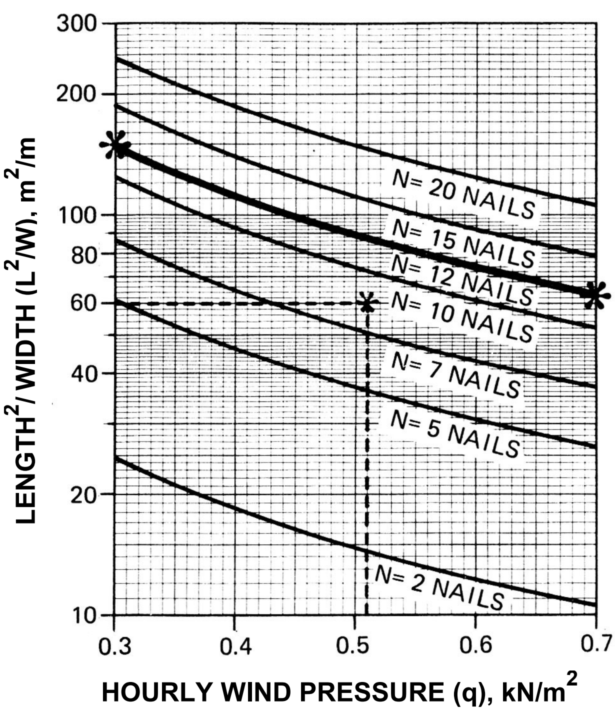


Figure 15 Edge-purlin splice nailing, wall height H = 4.8 m

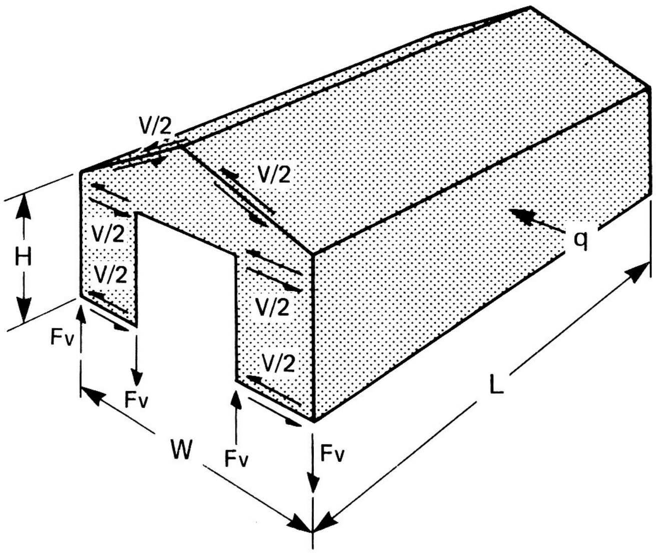


Figure 16 Endwall with large doorway

If the endwalls have no big doors or other major openings, a steel cladding and fastening schedule similar to the roof will be safe enough. If however there are large end doorways that reduce the effective width of the endwalls, special steps must be taken. Figure 16 shows where the endwall with doors may be weak. The remaining parts of the endwall (beside the doorway) will probably need extra reinforcing for both shear and moment. A detailed analysis of the endwall design is beyond the scope of this plan because of the variety of openings and dimensions possible. However, in principle the remaining endwall parts should be designed as short cantilever beams, for both shear and moment. Extra shear stress due to reduced effective width can be easily handled by adding interior cladding such as plywood or flakeboard, well-nailed for shear. The vertical forces F due to moment will probably require special tension connections at the bottom corners of the shortened endwalls. With pole-framed endwalls these uplift forces can be handled by anchoring the poles at the corners and door-jambs in concrete backfill. With stud-framed endwalls, special hold-down anchors will be needed to hold corner posts and doorjamb studs down to the concrete foundation.

EXAMPLE PROBLEM Design a roof-endwall steel diaphragm system for wind-bracing a stud-frame implement shed 15 x 30 m, 4.8 m high from foundation to roof, for High River, Alberta (1/10 hourly wind pressure, q = 0.51 kN/m^2).

For the roofing stitch-screw spacing (Figure 14), L/W = 30/15 = 2. Entering Figure 14 at L/W = 2 and q = 0.51, the maximum screw spacing is 200 mm o.c. With roof purlins spaced at 600 mm, this calls for two extra stitch-screws between each pair of purlins. Similarly, screws on the flats of the roofing will be required at 200 mm o.c. where the roofing meets the blocking ⑤ at the endwall gable trusses (Figure 4). If there are any roofing sheet end-laps, one screw at each rib will be adequate as long as the rib spacing is not greater than 200 mm. Similarly at the eaves and ridge where the roofing sheets meet the edge purlins, one screw beside each rib will transfer shear to the spliced edge purlins.

For the edge purlin connection (⑨, Figure 4), L^2/W = 30^2/15 = 60. Entering Figure 15 at L^2/W = 60 and q = 0.51kN/m^2, 10 concrete nails are required for each half-joint. The steel straps will be 100 mm wide (under 12 nails, see*), and the straps will have to be long enough to hold five rows of nails in each half-joint (alternating two and three nails per row), plus endspaces, making 500 mm long. Or you can use the table on page 4.

Assuming doorways 4.8 m wide are centered in each endwall (Figure 16), the effective width of the endwall reduces to (15 - 4.8) = 10.2 m. This changes the stitch-screw spacing to (10.2/15) x 200 mm = 136 mm. Since this spacing is closer than the 150 mm limit for no. 8 screws and 0.3 mm (30 gauge) steel, the endwalls need some extra shear reinforcing such as plywood or aspen flakeboard interior cladding. Criss-crossed wind bracing is another alternative, but with this it is not easy to make the end connections strong enough.

In this example the vertical forces F_V need special consideration over and above the normal anchor bolts used to connect sill to foundation. The endwall bending moments resulting from the roof shear force V must be balanced by the two hold-down forces F_V such that

$$HV/2 = F_V\,(15 - 4.8)/2,$$

$$\text{therefore } F_V = \frac{HV}{10.2}$$

$$\text{but } V = \frac{1.0 \times 0.8}{4}\,qHL = (0.45)(0.51)(4.8)(30) = 33 \text{ kN}$$

$$\text{therefore } F_V = (4.8)(33)/10.2 = 15.5 \text{ kN}$$

Tension connections to resist F_V = 15.5 kN are required at four locations in the endwall with the doorway (assuming the wind may blow from either side). These connections will be in addition to the normal frame-to-foundation connections required to resist roof uplift and horizontal shear.

REFERENCES

Turnbull, J.E. 1983. A summary of diaphragm design for wind bracing in stud-wall farm buildings. ASAE Trans. Vol. 26, No. 2, pp 549-556, 561

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