COLD-FORMED STEEL FRAMING STRUCTURAL CALCULATION REPORT

LGS SAMPLE HOUSE

Prepared by	: Structomat
Date	: 2026-11-18

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1 INTRODUCTION

1.1 Scope

This report covers the structural design of the cold-formed steel (light gauge steel) framing of a single-storey house: wall studs and tracks, box headers over openings, roof trusses and their heel and ridge connections, and the lateral load resisting system (roof diaphragm and sheathed shear walls). Loads and load combinations follow NBC 2020 Div. B Part 4; members and connections are designed by Limit States Design to AISI S100-16 (R2020) w/S3-22 + Appendix B and AISI S240-20.

The building is a sample project prepared to demonstrate the calculation. Site-dependent input (location, terrain, site class, dead load build-ups) are assumptions and are identified as such in Section 4.

1.2 Description of the Structure

Plan dimensions are 12.192 m × 9.144 m out-to-out of the wall framing; the wall height is 2.743 m. The roof is a gable roof with a slope of 6:12 (26.6°), the ridge running parallel to the long side, with an eave overhang of 406 mm. The ground floor is a concrete slab on grade. Location for climatic data: Barrie, Ontario.

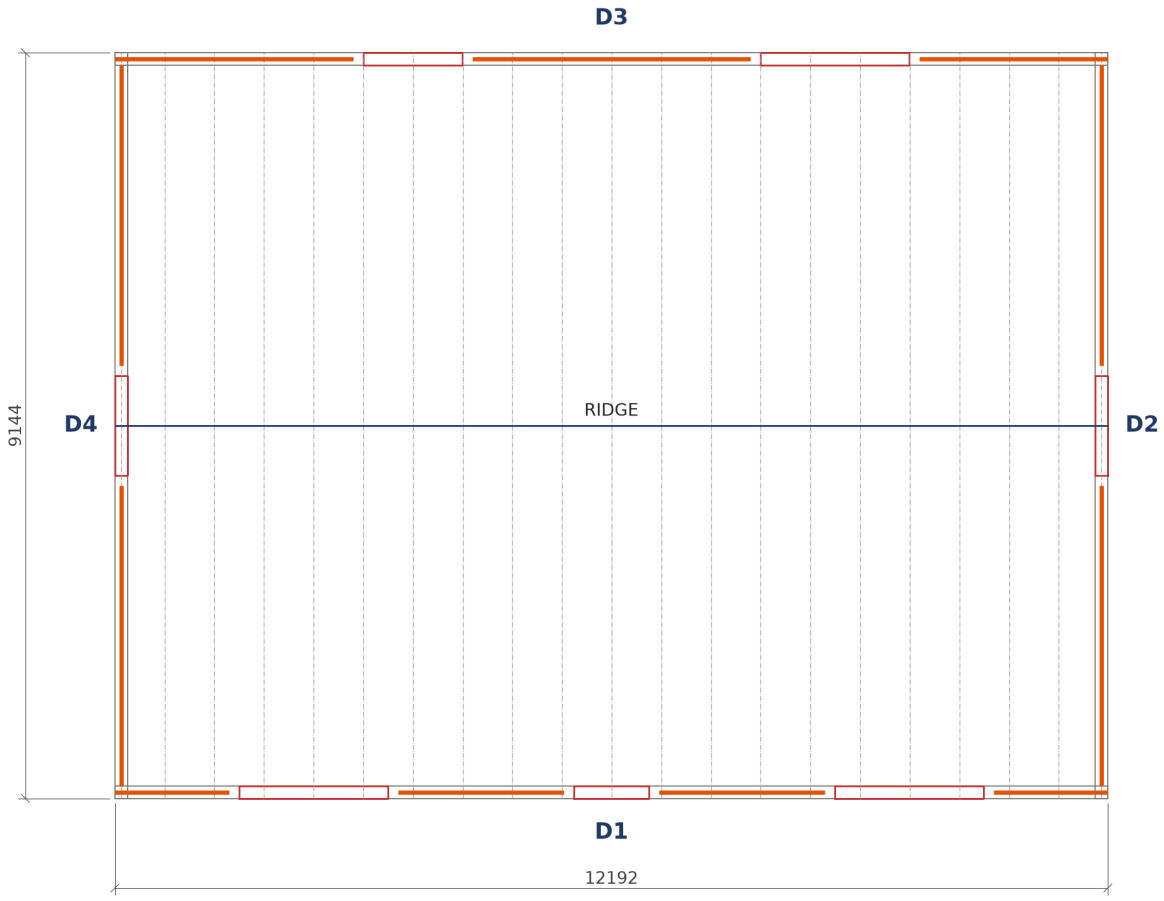
1.3 Structural System

Gravity loads: Fink roof trusses at 610 mm centres span 9.144 m between the two eave walls and bear on the top tracks directly above the wall studs (in-line framing, stud spacing 610 mm). Over openings the truss reactions are carried by box headers to jack and king studs. All walls bear on the slab through the bottom track.

Lateral loads: the roof sheathing (OSB 11 mm, blocked) acts as a diaphragm spanning between the exterior walls. The exterior walls are sheathed on the outside with OSB 11 mm panels fastened with #8 screws at 150 mm (panel edges) / 300 mm (field) and act as Type I shear wall segments between openings, with hold-downs at the segment ends. Wall and roof OSB: 11 mm (7/16 in) rated sheathing, panel mark 1R24/2F16/W24 (the grade of the AISI S240 Canadian shear wall table), all panel edges blocked.

Table 1 – Framing members

Member	Section (SSMA designation)	Fy / Fu (MPa)
Truss top chord	550S162-97	345 / 450
Truss bottom chord	550S162-68	345 / 450
Truss webs	350S162-43	345 / 450
Wall studs, king / jack / cripple studs	600S162-43	230 / 310
Wall tracks	600T125-43	230 / 310
Box header (2 members)	800S162-68	345 / 450



PLAN - walls D1-D4, openings (red), roof trusses @ 610 mm (dash-dot), Type I shear wall segments (orange)

***Figure 1** – Plan — walls, openings, truss lines and shear wall segments*

2 CODES AND REFERENCES

Table 2 – Codes and references

Document	Used for
National Building Code of Canada 2020, Div. B Part 4	Loads, importance factors, load combinations, lateral deflection limit
AISI S100-16 (R2020) w/S3-22 + Appendix B	Member and connection resistance — Limit States Design (LSD). Screw connections (J4): base AISI S100-16 (R2020) = CSA S136-16 resistance factors (φ = 0.40), the S3-22 screw factors are not used
AISI S240-20	Cold-formed steel framing: studs, headers, coped truss heel, gusset plates, shear walls, diaphragms, bridging
SSMA Product Technical Guide (2017)	Section designations, dimensions, design thickness, steel grades; generic screw shear strength Pss (p. 70, after CFSEI Tech Note F701-12)
NRCan 2020 NBC Seismic Hazard Tool	Seismic hazard values Sa(T) for the site
AISI S230-15, Prescriptive Method for One- and Two-Family Dwellings	Reference for the full-height gable endwall system (Section E3, Table E3-17); member sizes here are calculated, not taken from the tables
Second analysis program	3D check of the gravity load path, the lateral distribution, the shear walls (shell model) and the gable end framing (Section 9)

3 MATERIALS AND SECTIONS

Wall studs and tracks: Fy = 230 MPa, Fu = 310 MPa. Roof truss members and box headers: Fy = 345 MPa, Fu = 450 MPa. Modulus of elasticity E = 203000 MPa. Resistance factors (LSD): compression φc = 0.80, bending φb = 0.90, tension φt = 0.90 (yielding) / 0.75 (rupture). Truss connections use #10 self-drilling screws; wall sheathing #8 screws. Screw connections (Section J4) use the resistance factor of the base AISI S100-16 (R2020) edition, which corresponds to CSA S136-16 referenced by NBC 2020: φ = 0.40 (LSD) for all screw limit states; the separate screw factors of Supplement S3-22 are not used.

Table 3 – Section dimensions and gross properties

Section	Depth (mm)	Flange (mm)	Lip (mm)	Design t (mm)	Inside radius (mm)	Gross area (mm²)	Mass (kg/m)
600S162-43	152.4	41.3	12.7	1.146	1.81	288	2.26
600T125-43	154.7	31.8	–	1.146	1.81	245	1.92
800S162-68	203.2	41.3	12.7	1.811	2.72	539	4.23
550S162-97	139.7	41.3	12.7	2.583	3.87	590	4.63
550S162-68	139.7	41.3	12.7	1.811	2.72	424	3.33
350S162-43	88.9	41.3	12.7	1.146	1.81	216	1.69

S = lipped C-section (stud / joist), T = track: the depth in the SSMA designation of a track is the inside dimension, the depth in the table is out-to-out (designation + 2t). Effective section properties are calculated by the effective width method; wall studs are checked at the standard web punchout (38 × 102 mm), truss members are unpunched.

4 LOADS

4.1 Site and Climatic Data

Table 4 – Site data

Item	Value	Source
Location	Barrie, Ontario	NBC 2020 Div. B Appendix C, Table C-2
Ground snow load Ss / associated rain load Sr (1/50)	2.5 / 0.4 kPa	same table
Hourly wind pressure q (1/10, 1/50)	0.28 / 0.36 kPa	same table
Importance category	Normal	NBC 2020 Table 4.1.2.1 — assumed (dwelling)
Terrain	open terrain	assumed (conservative)
Site designation (seismic)	X450	assumed — no geotechnical report

4.2 Dead Loads

Table 5 – Dead loads (assumed build-ups)

Load	kPa	Applied on
Roof — top chord level (roofing, sheathing, truss self-weight share)	0.50	plan projection
Roof — bottom chord level (ceiling board, insulation, services)	0.35	plan projection
Exterior wall (cladding, sheathing, framing, insulation, gypsum board)	0.45	wall surface

Check of the truss self-weight included in these values (from the material take-off, over the roof plan area 121.4 m² incl. overhangs): top chords + webs 1253 kg → 0.101 kPa, leaving 0.40 kPa for roofing and roof sheathing (e.g. 11 mm OSB ≈ 0.07 kPa + roofing); bottom chords 578 kg → 0.047 kPa, leaving 0.30 kPa for ceiling board, insulation and services. The assumed dead loads remain adequate for the final truss sections.

4.3 Live Loads

Roof live load 1.0 kPa (NBC 2020 Table 4.1.5.3. — Roofs 1.0 kPa, Attics w/ limited accessibility 0.5 kPa); attic with limited accessibility 0.5 kPa on the bottom chord. The roof live load is not combined with snow.

4.4 Snow Load

S = Is·[Ss·(Cb·Cw·Cs·Ca) + Sr] — NBC 2020 4.1.6.2. + Table 4.1.6.2.-A; gable roof: NBC 2020 4.1.6.9. Gable Roofs.

Table 6 – Snow load

Parameter	Value
Importance factor Is (ULS / SLS)	1.0 / 0.9
Basic roof snow load factor Cb	0.80
Wind exposure factor Cw	1.00
Slope factor Cs	1.00
Balanced load: Ca	1.00
Balanced snow load S (ULS / SLS)	2.40 / 2.16 kPa
Unbalanced load: Ca leeward / windward	1.25 / 0.00
Unbalanced snow load S, leeward / windward slope (ULS)	2.90 / 0.00 kPa

4.5 Wind Load

External pressure p = Iw·q·Ce·Ct·Cg·Cp — NBC 2020 4.1.7.3. + Table 4.1.7.3. Low-rise building coefficients CgCp for the main structural system: NBC 2020 Figure 4.1.7.6.-A + Table 4.1.7.6. Reference height 6.0 m, Ce = 0.903, Ct = 1.0, Iw = 1.0 (ULS) / 0.75 (SLS), Iw·q·Ce·Ct = 0.325 kPa. End-zone widths: z = 1.0 m, y = 6.0 m.

Table 7 – Main structural system — Load case A — wind perpendicular to the ridge

Building surface	1	1E	2	2E	3	3E	4	4E
CgCp	1.03	1.37	-0.18	-0.36	-0.83	-1.10	-0.73	-1.00
p (kPa)	0.336	0.445	-0.060	-0.117	-0.271	-0.359	-0.239	-0.326

Table 8 – Main structural system — Load case B — wind parallel to the ridge

Building surface	1	1E	2	2E	3	3E	4	4E	5	5E	6	6E
CgCp	-0.85	-0.90	-1.30	-2.00	-0.70	-1.00	-0.85	-0.90	0.75	1.15	-0.55	-0.80
p (kPa)	-0.276	-0.293	-0.423	-0.650	-0.228	-0.325	-0.276	-0.293	0.244	0.374	-0.179	-0.260

Internal pressure: Cgi = 2.0, Cpi = -0.45 to +0.30 (NBC 2020 Table 4.1.7.7.; Cgi 4.1.7.3.(10)) → pi = -0.293 / +0.195 kPa. Specified base shear: 31.6 kN (case A), 17.0 kN (case B).

Secondary members (studs, individual trusses for local effects) — NBC 2020 Figure 4.1.7.6.-B (values read from the chart); NBC 2020 Figure 4.1.7.6.-E (values read from the chart). Net stud pressures including internal pressure: +0.84 kPa (pressure), -0.77 kPa (suction, interior zone), -0.85 kPa (suction, end zone), for a tributary area of 1.67 m².

4.6 Seismic Load

Hazard values for site designation X450 (2 % in 50 years): Sa(0.2) = 0.208, Sa(0.5) = 0.131, Sa(1.0) = 0.074, Sa(2.0) = 0.0359, Sa(5.0) = 0.00966, Sa(10.0) = 0.00346. IE = 1.0, Fs = 1.0; IE·Fs·Sa(0.2) = 0.208 → seismic design is required (NBC 2020 4.1.8.1.(2)–(15)). Equivalent Static Force Procedure (4.1.8.11.): NBC 2020 Table 4.1.8.4.-C, Table 4.1.8.5.-B, 4.1.8.7., Table 4.1.8.9., 4.1.8.11. + Table 4.1.8.11.

Choice of the seismic force resisting system: the site has S(0.2) = 0.208 > 0.12. AISI S240-20 A1.2.2(b) requires the additional seismic provisions of AISI S400 in Canada where S(0.2) > 0.12 and RdRo ≥ 1.56. These provisions (capacity design, detailing) are not applied in this design; the shear walls are therefore classified as 'Other cold-formed SFRSs not defined above' of Table 4.1.8.9. with Rd = 1.0 and Ro = 1.0 (RdRo < 1.56), height limit 15 m for SC2. No upper limit of 4.1.8.11.(2)(c) applies because Rd < 1.5.

Table 9 – Equivalent static seismic force

Parameter	Value
Seismic force resisting system	Other cold-formed SFRSs not defined above
Rd / Ro	1.0 / 1.0
Seismic category	SC2
Building height hn	4.16 m
Fundamental period Ta	0.146 s
S(Ta)	0.208
Higher mode factor Mv	1.0
Seismic weight W (roof dead + upper half of walls + gable triangles + 25 % snow)	211.7 kN
V by formula / upper limit / lower limit	44.04 / – / 2.82 kN
Design base shear V	44.04 kN (V/W = 0.2080)

Comparison of factored lateral loads: wind 1.4W = 44.2 kN (perpendicular to ridge) / 23.7 kN (parallel to ridge); earthquake 1.0E = 44.0 kN. Per line of resistance (Section 5.4) earthquake governs direction A and earthquake governs direction B; every line is nevertheless designed for both actions with their own resisting dead load (Section 8.1).

4.7 Load Combinations

Table 10 – Ultimate limit state combinations — NBC 2020 Table 4.1.3.2.-A + Sentences 4.1.3.2.(3),(5)

Case	Combination
1	1.40D
2a	1.25D + 1.50L
2b	1.25D + 1.50L + 1.00S
2c	1.25D + 1.50L + 0.40W
2d	0.90D + 1.50L
3a	1.25D + 1.50S
3b	1.25D + 1.00L + 1.50S
3c	1.25D + 1.50S + 0.40W
3d	0.90D + 1.50S
4a	1.25D + 1.40W
4b	1.25D + 0.50L + 1.40W
4c	1.25D + 0.50S + 1.40W
4d	0.90D + 1.40W
5a	1.00D + 1.00E
5b	1.00D + 0.50L + 0.25S + 1.00E

Serviceability: snow with Is = 0.9, wind multiplied by 0.75 (importance factor for SLS), unfactored dead and live loads.

4.8 Load Assumptions

1) Cw = 1.0 (4.1.6.2.(3); the 0.75 reduction for exposed rural sites was NOT used).

2) Roof surface assumed NOT slippery → Cs per 4.1.6.2.(5).

3) Internal pressure Category 2: Cpi −0.45 … +0.30 (Table 4.1.7.7.).

4) Roof dead loads (top / bottom chord) are in kPa of plan projection.

5) Roof live load 1.0 kPa is NOT combined with snow (4.1.5.5.(2)); L in snow combinations = attic 0.5 kPa on the bottom chord.

6) Main-system wind: eave soffit pressure and the wall strip above the heel are not taken separately (He = wall + heel).

7) Load case B: the gable end-zone (5E / 6E) area share is taken as z / W.

8) Secondary member tributary area = length × spacing (no length/3 effective width increase — conservative).

9) Seismic: regular structure assumed (rectangular plan, single storey) → Equivalent Static Force Procedure, 4.1.8.7.(1).

10) Seismic: RdRo < 1.56 → by AISI S240-20 A1.2.2(b) the additional provisions of AISI S400 do not apply; shear walls and diaphragm are designed to AISI S240 and S100 (Table 4.1.8.9 'Other cold-formed SFRSs not defined above', Rd = Ro = 1.0).

5 STRUCTURAL ANALYSIS

5.1 Roof Truss Model

Each truss is analysed as a plane frame with 99 load arrangements (combinations × snow patterns × wind cases, directions and internal pressure signs). Truss classes: 1 interior-zone, 18 end-zone (wind) and 2 gable end trusses; member design uses the envelope of the interior and end-zone classes.

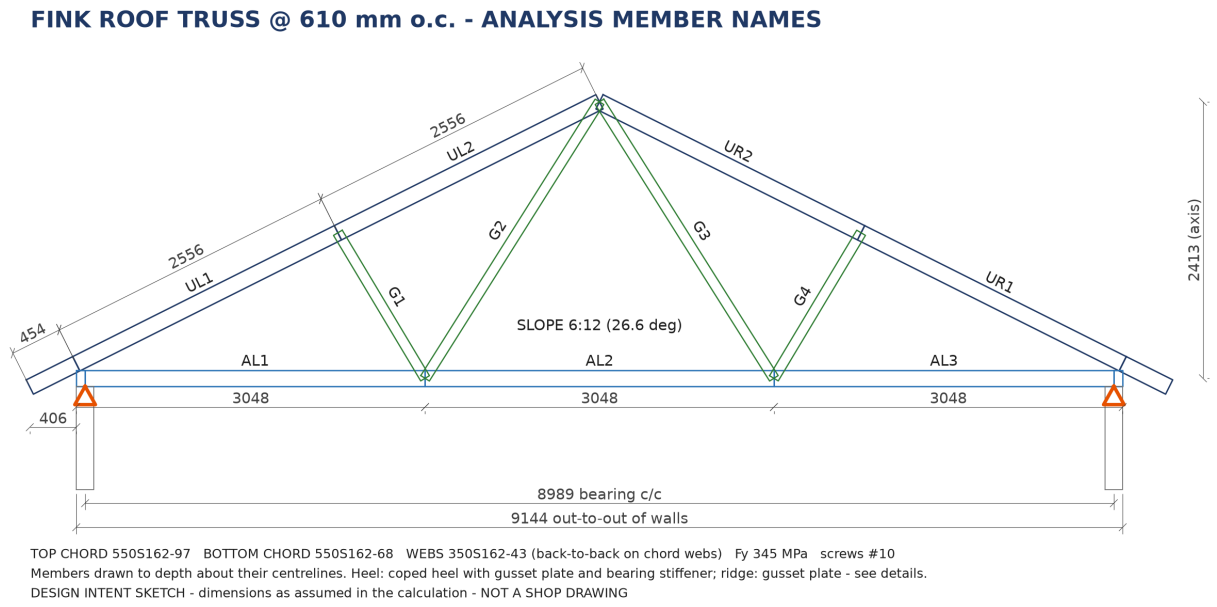


Figure 2 – Roof truss — geometry and analysis member names

Table 11 – Truss line loads per horizontal metre (truss spacing 0.610 m)

Load	kN/m
Dead — top chord	0.305
Dead — bottom chord	0.213
Attic live — bottom chord	0.305
Roof live — top chord	0.610
Balanced snow (ULS) — top chord	1.464
Unbalanced snow, leeward slope (ULS) — top chord	1.769

Table 12 – Truss support reactions per basic load case, end-zone truss (unfactored; + upward)

Load case	Left (kN)	Right (kN)
D — dead	2.49	2.49
L — attic live (bottom chord)	1.39	1.39
L — roof live	3.04	3.04
S — balanced snow	7.29	7.29
S(L) — unbalanced snow, heavier on left	6.84	1.96
S(R) — unbalanced snow, heavier on right	1.96	6.84
W-A(L) — wind case A from left	-0.57	-0.88
W-A(R) — wind case A from right	-0.88	-0.57
W-B(L) — wind case B, zone 2 on left	-1.69	-1.27
W-B(R) — wind case B, zone 2 on right	-1.27	-1.69
internal pressure (+)	-0.54	-0.54
internal pressure (−)	0.82	0.82

5.2 Truss Member Forces

Table 13 – Factored member force envelope — end-zone truss (N: − compression / + tension)

Member group	N min (kN)	Load case	N max (kN)	Load case	|M| max (kNm)	|V| max (kN)
Bottom chord — middle panel (AL2)	-0.14	4d · W-B(L)+	12.84	3b · S	0.53	1.15
Bottom chord — heel zone	-0.56	4d · W-B(R)+	18.82	3b · S	1.20	14.24
Bottom chord — end panel (AL1, AL3)	-1.25	4d · W-B(L)+	18.82	3b · S	1.20	1.34
Short web (G1, G4)	-6.31	3c · S(R) · W-A(R)−	1.42	4d · W-B(R)+	0.00	0.00
Long web (G2, G3)	-0.72	4d · W-B(R)+	7.81	3b · S(L)	0.00	0.00
Heel lap (rigid link)	-14.19	3b · S	1.16	4d · W-B(R)+	2.29	18.82
Top chord — ridge panel (UL2, UR2)	-19.71	3b · S	1.52	4d · W-B(R)+	1.48	3.60
Top chord — eave overhang (UL0, UR0)	-0.00	2d	0.55	3b · S(L)	0.25	1.10
Top chord — heel panel (UL1, UR1)	-22.72	3b · S	1.35	4d · W-B(R)+	2.50	3.62

Support reactions (factored): maximum 15.44 kN (3b · S), minimum -0.88 kN (4d · W-B(R)+; negative = uplift). Eccentricity of the chord centreline intersection from the bearing centreline: 331 mm.

Load case labels: combination number (Table 4.1.3.2.-A case) · snow pattern (S = balanced, S(L) / S(R) = unbalanced with the heavier load on the left / right slope) · wind (W-A / W-B = NBC load case A / B, (L) / (R) = wind from the left / right, + / − = sign of the internal pressure).

5.3 Wall Studs and Headers

Table 14 – Stud forces (length 2.741 m, spacing 0.610 m; factored; full-height gable wall studs in Section 7.4)

Stud class	P max (kN)	P min (kN)	M max (kNm)	V max (kN)
Eave wall — interior zone	15.91	0.26	0.675	0.985
Eave wall — end zone	15.91	-0.54	0.683	0.997

Table 15 – Headers — factored actions (x = start of opening from wall start)

Wall	Opening	x (mm)	Span (m)	q max (kN/m)	M (kNm)	V (kN)	Jack P (kN)	King w (kN/m)
D1	window	1524	1.912	25.66	12.89	24.53	24.53	1.45
D1	door	5639	0.997	25.66	3.90	12.79	12.79	0.91
D1	window	8839	1.912	25.66	12.89	24.53	24.53	1.45
D2	window	3962	1.302	3.54	0.75	2.31	2.31	see 7.4
D3	door	2438	1.912	25.66	11.72	24.53	24.53	1.45
D3	window	7925	1.302	25.66	6.56	16.70	16.70	1.09
D4	window	3962	1.302	3.54	0.75	2.31	2.31	see 7.4

q max = roof load on the wall top (eave walls: truss reactions / truss spacing; gable walls: the end roof strip, half the truss spacing, carried down by the cripple studs) + the self-weight of the wall above the header (1.25 × 0.45 kPa × height from the top of the opening to the top of the cripple studs; full-height gable walls 1.80 + 1.74 kN/m over 3.10 m). Uplift (q min) is taken without the wall self-weight.

5.4 Lateral Load Path

Table 16 – Lateral loads to the lines of resistance

Direction	W base shear (kN)	Roof share	Line shear 1.4W (kN)	Line shear 1.0E (kN)	Governs	Roof shear (kN/m)	Roof aspect ratio
A — wind perpendicular to ridge	31.6	0.64	15.09	26.43	earthquake	2.89	1.33
B — wind parallel to ridge	16.8	0.67	8.07	27.96	earthquake	2.29	0.75

W base shear here is the integral of the wind line load along the diaphragm span (end zones placed at their actual positions and with the actual wall height under the gable). In Section 4.5 the end-zone share of the gable area is taken as the width ratio z/W, which is conservative because the gable is lowest at the eaves; this explains the small difference in direction B. Roof shear and line shears are the envelope of 1.4W and 1.0E.

5.5 Analysis Assumptions

1) Truss: linear (first-order) 2D frame; chords continuous, pinned at the ridge; web members pinned (S240-20 E4.3). P-δ amplification is applied in member design through B1 (S100 Eq. C1.2.1.1-3).

2) Heel: coped heel with gusset plate (S240 E4.6.2), modelled with a rigid link between the chord centrelines (the plate transfers moment to both chords); the residual eccentricity moment is in the analysis and the connection is designed for these forces.

3) Supports: on the wall centrelines, left pinned / right roller (horizontal reaction at wall top goes to the diaphragm).

4) Wind on trusses: NBC Figure 4.1.7.6.-A load cases A and B roof pressures, both directions, internal pressure ± applied to the roof plane (vented attic / ceiling not airtight assumed); eave soffit pressure is NOT included.

5) End-zone trusses: trusses within distance y of either end along the ridge (each corner is windward in turn, Figure Note 1).

6) Stud: simply supported; axial = truss reaction + wall weight above mid-height; bending = the larger secondary-member net pressure (Figure 4.1.7.6.-B) with the same combination factor (wind direction not matched — conservative).

7) Header: truss reactions as an equivalent uniform load (R / spacing) plus the self-weight of the wall above the header (Table 15 note); the king stud takes the opening wind as a uniform load.

8) Lateral: flexible diaphragm, lines of resistance = exterior walls; roof share = total − lower half of the walls; each line is designed for the larger end reaction.

9) Seismic share per line 0.6·V (flexible diaphragm with the mass shifted ±0.1·D in place of the accidental torsion of 4.1.8.11.(9)). Each line is solved separately for 1.4W (resisting 0.9D less concurrent roof uplift) and 1.0E (resisting 1.0D) and the envelope is used.

10) Shear walls: Type I segments (S240-20 B5.2), chord force C = V·h/L; resisting load = 0.9D − concurrent 1.4W roof uplift (half the segment length).

11) Distribution within a line: segment share = max(resistance share, secant stiffness share per S240 Eq. B5.2.5-1; S240 has no distribution rule, the envelope is conservative); each segment ratio is checked as well.

12) The 70 % reduction for foundations (Figure 4.1.7.6.-A, Note 2) was NOT applied.

13) Full-height gable walls: there is no gable truss (the roof edge bears on the rake track); the gable wall studs are not part of the 2D wall analysis but are designed with their actual lengths.

6 ROOF TRUSS DESIGN

6.1 Chords

Chords are checked for combined axial force and bending at every analysis station and load arrangement. Compression: AISI S240 E4.4.2 with the effective lengths of E4.4.2.1 and the AISI S100 H1.2 interaction, with the P-δ amplifier B1 = Cm / (1 − αP/Pe1) ≥ 1.0 (AISI S100 Eq. C1.2.1.1-3, α = 1.0 for LSD); tension + bending: S100 H1.1; bending + shear: S100 H2-1. Bending resistance Mr is for the compression flange braced by sheathing or unbraced (lateral-torsional buckling), whichever applies at the station.

Table 17 – Top chord 550S162-97 — governing check per member

Member	KxLx (mm)	KtLt (mm)	Pr (kN)	Tr (kN)	Mr (kNm)	N (kN)	M (kNm)	B1	Eq.	Ratio	M + V ratio	Load case
UL0	908	908	113.9	183.2	7.19	0.55	0.25	1.00	H1.1	0.04	0.04	3b · S(L)
UL1	1917	1917	67.5	183.2	7.19	-22.72	2.50	1.00	H1.2	0.68	0.35	3b · S
UL2	1917	1917	67.5	183.2	7.19	-18.39	1.33	1.00	H1.2	0.46	0.21	3b · S
UR2	1917	1917	67.5	183.2	7.19	-18.39	1.33	1.00	H1.2	0.46	0.21	3b · S
UR1	1917	1917	67.5	183.2	7.19	-22.72	2.50	1.00	H1.2	0.68	0.35	3b · S
UR0	908	908	113.9	183.2	7.19	0.55	0.25	1.00	H1.1	0.04	0.04	3b · S(R)

Table 18 – Bottom chord 550S162-68 — governing check per member

Member	KxLx (mm)	KtLt (mm)	Pr (kN)	Tr (kN)	Mr (kNm)	N (kN)	M (kNm)	B1	Eq.	Ratio	M + V ratio	Load case
AL1	2228	2228	31.0	131.7	5.09	18.82	1.20	1.00	H1.1	0.37	0.24	3b · S
AL2	2286	2286	30.0	131.7	1.33	10.45	0.52	1.00	H1.1	0.31	0.40	2b · S
AL3	2228	2228	31.0	131.7	5.09	18.82	1.20	1.00	H1.1	0.37	0.24	3b · S

Governing top chord member UL1: N = -22.72 kN, M = 2.50 kNm (compression flange unbraced (LTB)) → 0.336 + 0.348 = 0.684 ≤ 1.0 (H1.2, load case 3b · S).

6.2 Concentrated Load at Top Chord Panel Point

AISI S240 E4.4.2.3 — compression web bearing on the top chord at node U4 (load case 3c · S(R) · W-A(R)−): P = 15.57 kN, M = 1.14 kNm, web reaction R = 6.33 kN; Pno = 174.8 kN, Mnxo = 7.99 kNm, Rn = 36.41 kN (bearing length 88.9 mm). Interaction 0.405 ≤ 1.192 → ratio 0.34.

6.3 Web Members

Compression webs: AISI S240 E4.4.4.1 (webs connected back-to-back to the chord webs; flexural buckling with the eccentricity of the connection about the weak axis). Tension: S100 Chapter D (yielding and rupture). End connections: at least four screws per end.

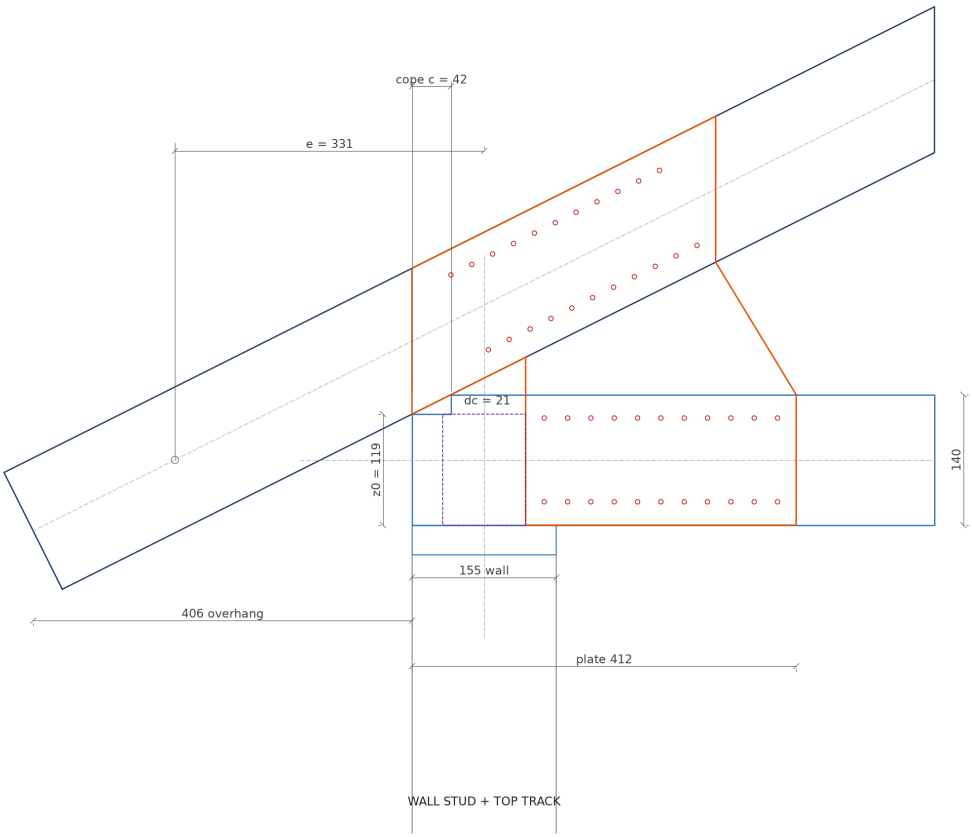
Table 19 – Web members 350S162-43 (e = 13.9 mm, φb·Mny = 0.562 kNm)

Member	L (mm)	L/r	R	φPn (kN)	N comp. (kN)	Ratio	Tr (kN)	N tens. (kN)	Ratio	End screws	Ratio
G1	1481	95	0.60	27.0	-6.34	0.25	66.9	1.41	0.02	4 × #10	0.59
G2	2854	183	0.74	9.0	-0.72	0.07	66.9	7.81	0.12	4 × #10	0.73
G3	2854	183	0.74	9.0	-0.72	0.07	66.9	7.81	0.12	4 × #10	0.73
G4	1481	95	0.60	27.0	-6.34	0.25	66.9	1.42	0.02	4 × #10	0.59

6.4 Heel Joint

The heel is a coped heel connection (AISI S240 E4.6.2, Commentary Figure C-E4.6.2-1). The chords lie in one plane; the end of the bottom chord has a rectangular cope (flange, lip and top of web removed) in which the top chord sits; the bottom edge of the top chord passes 119 mm above the wall top at the outer face (= db − dc) and meets the bottom chord top at the end of the cope. Length of cope c = 41.8 mm, depth of cope dc = 20.9 mm, flat web width h = 130.6 mm: c/h = 0.32 and dc/h = 0.16 are within the limits of E4.6.2.2 (0.10 < c/h < 1.0, 0.10 < dc/h < 0.4, h/t ≤ 200). A bearing stiffener 350S162-68 with Imin = 76451 mm⁴ ≥ 67.000 mm⁴ (about an axis parallel to the chord web) is provided, so the heel strength is the shear strength of the coped bottom chord (S100 G2) reduced by R = 0.976 − 0.556c/h − 0.532dc/h = 0.713 (Eq. E4.6.2-1).

The chords are connected by a heel gusset plate (t = 2.58 mm, Fy = 345 MPa) with a two-row screw group in each chord, designed for the top chord force plus the moment of the rigid heel link (elastic method). Plate compression: S240 E4.5.1 on the Whitmore width; plate tension: E4.5.2 (S100 Chapter D). The chord centreline intersection lies outside the bearing; the resulting moment is part of the truss analysis and the chords are designed for it.



TRUSS HEEL - COPED HEEL WITH GUSSET PLATE (AISI S240 E4.6.2, E4.5)
Chords in one plane. BOTTOM CHORD 550S162-68: rectangular cope at the end, top chord sits in it; cope c = 42 mm, dc = 21 mm (c/h 0.32, dc/h 0.16: within E4.6.2.2)
TOP CHORD 550S162-97: bottom edge 119 mm above the wall top at the outer face
HEEL GUSSET PLATE t = 2.58 mm, Fy 345 MPa, on the chord webs, approx. 412 x 438 mm, top edge along the top chord top face (not above the sheathing); E4.5.1 ratio 0.53
#10 screws: top chord 22 (11 x 2 rows @ 25), bottom chord 22 (11 x 2 rows @ 25); chord force 22.7 kN + link moment 2.29 kNm; ratio 0.92
BEARING STIFFENER 350S162-68 (Imin 76.451 mm4 >= 67.000), full remaining depth, screwed to the bottom chord web clear of the gusset plate screws (two plies only)
heel strength E4.6.2.1(a): G2 shear x R = 26.4 x 0.71 = 18.8 kN; reaction 15.44 kN; ratio 0.82
UPLIFT TIE truss to top track: 2 x #10 (tension 0.88 kN; ratio 0.33)
Chord centrelines meet 254 mm outside the outer wall face; e = 331 mm to bearing CL (moment in analysis, chords and screw groups designed for it)
DESIGN INTENT SKETCH - dimensions as assumed in the calculation - NOT A SHOP DRAWING

Figure 3 – Heel joint — design intent sketch

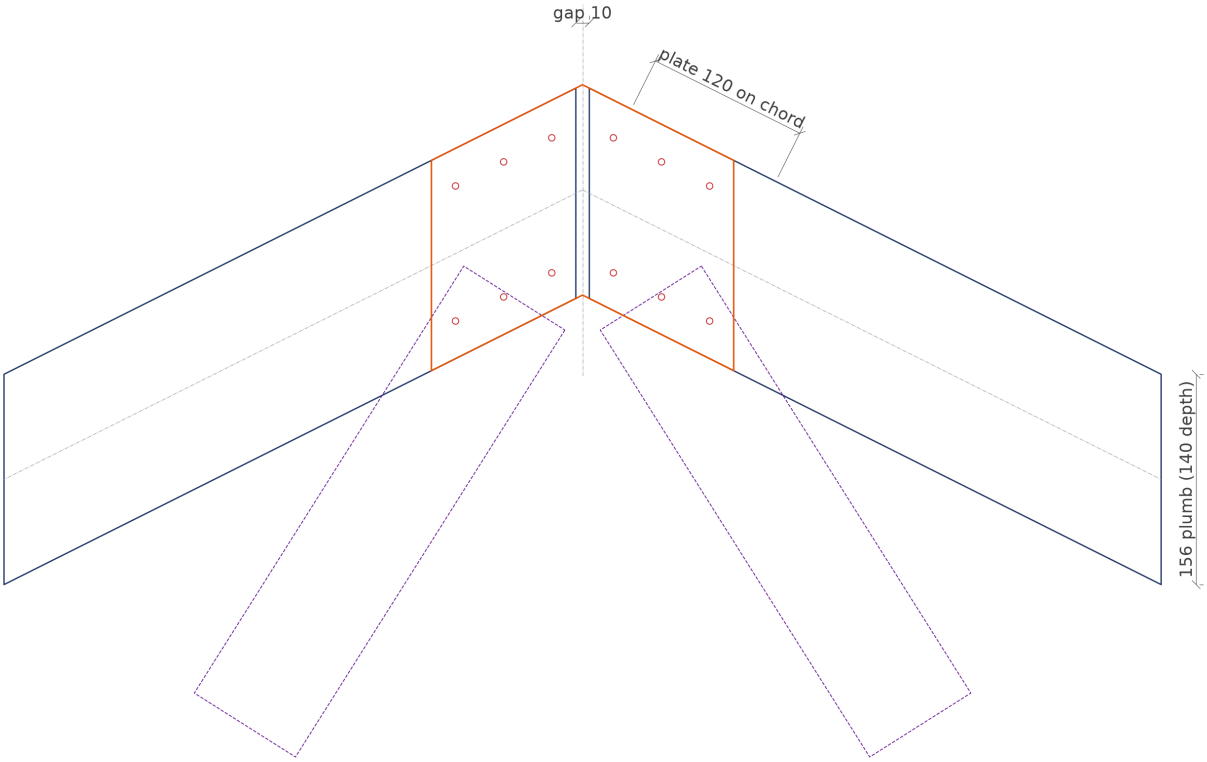
Table 20 – Heel joint checks

Check	Demand	Resistance	Ratio
Screw group, plate to top chord (22 × #10, 2 rows @ 25 mm, rows 90 mm apart)	22.72 kN + 2.29 kNm → 2.71 kN / screw	2.93 kN / screw	0.92
Screw group, plate to bottom chord (22 × #10, 2 rows @ 25 mm, rows 90 mm apart)	22.72 kN + 2.29 kNm → 2.71 kN / screw	2.93 kN / screw	0.92
Gusset plate t = 2.58 mm (W = 140 mm, Leff = 50 mm), compression E4.5.1 / tension E4.5.2	22.72 kN	43.1 / 112.1 kN	0.53
Heel strength — coped heel S240 E4.6.2.1(a): G2 shear × R, bearing stiffener 350S162-68	15.44 kN	26.4 × 0.713 = 18.8 kN	0.82
Heel shear on top chord web	14.24 kN	48.3 kN	0.29
Uplift tie to top track (2 × #10, pull-out / pull-over)	0.88 kN	2.68 kN	0.33

For reference, the web crippling resistance of the bottom chord without stiffener at the support would be 9.89 kN. The bearing stiffener is fastened to the bottom chord web outside the gusset plate screw area (two-ply connections only).

6.5 Ridge Joint

Gusset plate in compression: AISI S240 E4.5.1. Plate thickness 2.58 mm, width Wmin = 139.7 mm, unbraced length Leff = 50 mm; 6 × #10 screws per chord. Chord compression 17.07 kN ≤ 43.13 kN (Rg = 1.00) → ratio 0.40; chord tension 1.52 kN ≤ 112.1 kN; screws ratio 0.97.



TRUSS RIDGE - GUSSET PLATE (AISI S240 E4.5.1)
TOP CHORDS 550S162-97: ends plumb cut, 10 mm gap
GUSSET PLATE t = 2.58 mm (97 mil), Fy 345 MPa, width W = 140 mm min., on chord webs
6 x #10 per chord: 2 rows @ 90 mm, pitch 40 mm, end distance 20 mm from the cut (rows parallel to the chord axis)
Unbraced plate length across the joint Leff = 50 mm
Chord compression 17.1 kN; plate resistance 43.1 kN; ratio 0.40; screws ratio 0.97
WEB MEMBERS G2 / G3 350S162-43 (dashed): back-to-back on the opposite face, min. 4 x #10 each end
DESIGN INTENT SKETCH - dimensions as assumed in the calculation - NOT A SHOP DRAWING

***Figure 4** – Ridge joint — design intent sketch*

6.6 Truss Deflection

Bearing span 8989 mm. Maximum vertical deflection at node B1: dead 0.72 mm, attic live 0.40 mm, snow (SLS) 1.91 mm. Snow + live = 2.30 mm = span/3901 ≤ span/360 → ratio 0.09.

7 WALL FRAMING DESIGN

7.1 Studs

Studs 600S162-43 (Fy = 230 MPa, length 2741 mm) are designed for axial compression plus wind bending (AISI S100 H1.2) as all-steel members braced by the bridging row. In Canada, sheathing-braced design requires a theory, tests or rational engineering analysis (AISI S240-20 B1.2.2.5), which is not provided here; the sheathing-braced ratio is therefore reported for information only. The bridging is designed in Section 7.2.

Table 21 – Stud resistances

Bracing	Ly = Lt (mm)	Pr (kN)	Governing mode	Mr (kNm)	Governing mode
Sheathing-braced	600	27.07	local	2.26	distortional
All-steel (bridging only)	1370	21.03	local	2.26	distortional

Table 22 – Stud checks — all-steel (bridging-braced) (governing combination)

Stud class	Comb.	P (kN)	M (kNm)	Ratio	Sheathing-braced (info)	Shear	Wind deflection (mm)	δ
Eave wall — interior zone	3c	14.70	0.193	0.79	0.63	0.11	1.45	L/1895
Eave wall — end zone	3c	14.62	0.195	0.79	0.63	0.12	1.46	L/1873

7.2 Bridging

Bridging channel 150U50-54 (Fy ≥ 230 MPa) at mid-height, through the stud punchouts and clipped to every stud. Brace force per stud: 2 % of the stud axial force (AISI S240 B3.4.1.2) = 0.318 kN, plus the torsional brace force of a C-section under wind 1.5·(m/d)·W (AISI S100 C2.2.1, Eq. C2.2.1-3; m = 17.0 mm, W = w·a, w = 0.728 kN/m, a = 1370 mm) → F1 = 0.485 kN per stud. The force accumulates along the channel between anchorages.

Table 23 – Bridging checks

Check	Demand	Resistance	Ratio
Channel compression resistance (E2, KL = stud spacing, flexural-torsional)	Fcre 71.6 MPa, Fn 62.8 MPa, Ae 83.6 mm²	4.20 kN	–
Accumulated force, anchorage every 6 studs	6 × 0.485 = 2.91 kN	4.20 kN	0.69
Clip to stud (2 × #10)	0.485 kN	2.81 kN	0.17

Bridging shall be anchored (blocking or strap to the structure) at least every 6 stud spaces (calculated maximum 8; six spaces adopted). Brace stiffness (AISI S100 C2.3.1): required βrb = 66.4 N/mm, provided by the channel at the stud farthest from an anchorage EA/(n·s) = 4637 N/mm (clip flexibility neglected) → ratio 0.014. The anchorage element itself (blocking / strap) is not designed in this report.

7.3 Openings — Headers, Jack and King Studs

Box headers: AISI S240 B3.3.2 (bending, shear, web crippling and their interaction), 2 jack studs and one king stud each side of every opening. On the eave walls the truss reactions reach the header as concentrated loads through the cripple studs (top track by lever rule); the header moment is therefore the larger of the equivalent uniform load moment and the concentrated-load moment (factor f in the table, found with the 3D check, Section 9). Where bearing web crippling at the header end is exceeded, a compact bearing stiffener (AISI S100 G7.1 = base F5.1) is fitted at each end of each header member so that the reaction stays on the jack studs. Stiffener 350S162-97, length 200 mm, FITTED to bear on both header flanges (S100 G7.1: the reaction is applied directly into the stiffener), compact (w/t limits met); Pr = 2 × 135.7 kN per header end; 5 × #10 screws per stiffener to the header web (shear transfer, Chapter J) at every opening with stiffeners (the largest required number is specified for all).

Table 24 – Openings — utilisation ratios (header 2 × 800S162-68 box)

Wall	Opening	x (mm)	Span (m)	f	M	V	M+V+ cripling	End connection	End	Defl.	Jack	King
D1	window	1524	1.912	1.10	0.83	0.47	0.99	stiffener G7.1	0.84	0.39	0.58	0.98
D1	door	5639	0.997	1.22	0.25	0.24	0.56	bearing	0.85	0.06	0.30	0.81
D1	window	8839	1.912	1.10	0.83	0.47	0.99	stiffener G7.1	0.84	0.39	0.58	0.98
D2	window	3962	1.302	1.00	0.05	0.04	0.09	bearing	0.15	0.00	0.05	0.85
D3	door	2438	1.912	1.00	0.75	0.47	0.99	stiffener G7.1	0.84	0.36	0.58	0.98
D3	window	7925	1.302	1.21	0.42	0.32	0.79	stiffener G7.1	0.95	0.14	0.40	0.83
D4	window	3962	1.302	1.00	0.05	0.04	0.09	bearing	0.15	0.00	0.05	0.85

7.4 Full-Height Gable Walls

The gable walls are balloon framed: the studs (600S162-43) run in one piece from the bottom track to a rake track under the roof sheathing, so there is no gable truss and no hinge at ceiling level (the system of AISI S230-15, Section E3 and Table E3-17, gable endwalls over 10 ft). The roof edge (half a truss spacing) bears on the rake track and is carried by the studs; the out-of-plane wind on the gable wall is carried by the studs spanning from the slab to the rake line, where the rake track, screwed continuously to the roof sheathing, delivers the reaction to the roof diaphragm. Each stud is designed as a simply supported all-steel member (AISI S100 H1.2) with its actual length, braced by bridging rows at the wall spacing (1372 mm; the governing unbraced length Ly = Lt is the longest bridging interval, Cb = 1), for 1.25D + 1.4W + 0.5S and 1.25D + 1.5S + 0.4W, with the secondary-member wind pressure of NBC Figure 4.1.7.6.-B for its own tributary area; axial load = roof edge strip + own wall weight above mid-height; deflection limit L/360 (AISI S230 uses L/240). At the gable window there are no studs above the opening, so the king studs collect half the window width each; they are doubled (back to back). The cripple studs above the header span from the header to the rake; the header carries their bottom reactions and the upper half of the window wind out of plane on its weak axis to the king studs.

Table 25 – Full-height gable wall studs — governing stud, king group and cripple per wall

Wall	Member	s (mm)	L (mm)	Trib. (mm)	Ly = Lt (mm)	P (kN)	M (kNm)	Pr (kN)	Mr (kNm)	Ratio	δ / (L/360)
D2	stud	3205	4618	622	1372	1.15	1.914	20.00	2.262	0.92	0.91
D2	king ×2	5305	4946	903	1372	1.76	3.587	39.01	4.523	0.85	0.91
D2	cripple above header	4425	2892	610	1520	0.84	0.758	19.33	2.262	0.38	0.23
D4	stud	3205	4618	622	1372	1.15	1.914	20.00	2.262	0.92	0.91
D4	king ×2	5305	4946	903	1372	1.76	3.587	39.01	4.523	0.85	0.91
D4	cripple above header	4425	2892	610	1520	0.84	0.758	19.33	2.262	0.38	0.23

Table 26 – Full-height gable walls — header and connections

Check	Demand	Resistance	Ratio
D2 header out of plane (span 1467 mm; window wind 0.733 kN/m + cripples 2.04 kN)	0.946 kNm	9.661 kNm	0.10
D2 header ↔ king studs, 2 × #10 per end (shear, out of plane)	2.581 kN	2.81 kN	0.92
D2 sill ↔ king studs, 2 × #10 per end (lower half of window + cripples below)	0.879 kN	2.81 kN	0.31
D2 sill track out of plane (strong axis)	0.448 kNm	1.654 kNm	0.27
D2 stud ↔ bottom track web crippling (S240 B3.2.5.1, C = 3.70, φ = 0.75)	1.658 kN	2.45 kN	0.68
D2 rake end clip angle, 2 × #10 per leg (shear; resultant of wind reaction and roof edge load)	1.842 kN	2.81 kN	0.66
D2 rake track bending between studs (span 708 mm along the slope, roof edge strip)	0.076 kNm	0.127 kNm	0.60
D2 rake track ↔ roof sheathing, screws @ 150 mm (steel side; OSB side with the diaphragm boundary, Section 8)	2.79 kN/m	–	0.32
D2 bottom track out-of-plane line load to the slab (anchorage demand)	2.79 kN/m	anchor design outside scope	–
D4 header out of plane (span 1467 mm; window wind 0.733 kN/m + cripples 2.04 kN)	0.946 kNm	9.661 kNm	0.10
D4 header ↔ king studs, 2 × #10 per end (shear, out of plane)	2.581 kN	2.81 kN	0.92
D4 sill ↔ king studs, 2 × #10 per end (lower half of window + cripples below)	0.879 kN	2.81 kN	0.31
D4 sill track out of plane (strong axis)	0.448 kNm	1.654 kNm	0.27
D4 stud ↔ bottom track web crippling (S240 B3.2.5.1, C = 3.70, φ = 0.75)	1.658 kN	2.45 kN	0.68
D4 rake end clip angle, 2 × #10 per leg (shear; resultant of wind reaction and roof edge load)	1.842 kN	2.81 kN	0.66
D4 rake track bending between studs (span 708 mm along the slope, roof edge strip)	0.076 kNm	0.127 kNm	0.60
D4 rake track ↔ roof sheathing, screws @ 150 mm (steel side; OSB side with the diaphragm boundary, Section 8)	2.79 kN/m	–	0.32
D4 bottom track out-of-plane line load to the slab (anchorage demand)	2.79 kN/m	anchor design outside scope	–

As shear walls, the full-height gable walls are taken with the height of the longest stud in the segment (4.93 m; conservative for the hold-down force C = V·h/w and the deflection, Section 8.1). The wall panel elevations (Section 12) show the stud lengths and the rake track.

8 LATERAL SYSTEM DESIGN

8.1 Shear Walls

Type I shear wall segments (AISI S240 B5.2): nominal shear resistance vn = 9.9 kN/m for the sheathing and fastener schedule used, φ = 0.70. Direction A = wind perpendicular to the ridge (resisted by the gable walls), direction B = wind parallel to the ridge (resisted by the eave walls).

Table 27 – Seismic force per line: flexible / rigid diaphragm envelope (1.0E incl. ±0.1·D accidental eccentricity)

Dir.	Wall	Line stiffness (kN/mm)	Flexible 0.6V (kN)	Rigid (kN)	1.0E design (kN)	1.4W (kN)	Line design V (kN)
A	D2	12.24	26.43	25.06	26.43	15.09	26.43
A	D4	12.24	26.43	25.06	26.43	15.09	26.43
B	D1	7.87	26.43	18.94	26.43	8.07	26.43
B	D3	14.63	26.43	27.96	27.96	8.07	27.96

V = 44.04 kN. Line stiffness is the sum of the segment secant stiffnesses (AISI S240 Eq. B5.2.5-1); centre of rigidity x = 6.096 m, y = 5.922 m. The stiffer eave wall D3 (fewer openings) attracts more than the flexible-diaphragm share under the rigid assumption; the larger value is used.

Table 28 – Shear wall lines (factored)

Dir.	Wall	V (kN)	Vr (kN)	Ratio	Chord C (kN)	Chord stud ratio	HD net tension (kN)	Base shear (kN/m)	Collector (kN)	Defl. (mm)	δ
A	D2	26.43	51.1	0.52	17.67	0.83	14.92	2.89	5.13	0.67	h/7330
A	D4	26.43	51.1	0.52	17.67	0.83	14.92	2.89	5.13	0.67	h/7330
B	D1	26.43	47.7	0.58	11.11	0.61	6.81	2.17	11.52	0.20	h/13614
B	D3	27.96	59.9	0.49	9.31	0.57	3.24	2.29	8.13	0.12	h/21960

Table 29 – Shear wall segments (positions from wall start)

Wall	Segment (mm)	Width (m)	h/w	Vr (kN)	V 1.4W (kN)	V 1.0E (kN)	Chord force C (kN)	Resisting load (kN)	Net HD tension (kN)	HD case
D2	155 – 3838	3.683	1.34	25.5	7.54	13.21	17.67	2.75	14.92	1.0E
D2	5305 – 8989	3.684	1.34	25.5	7.55	13.22	17.67	2.75	14.92	1.0E
D4	155 – 3838	3.683	1.34	25.5	7.54	13.21	17.67	2.75	14.92	1.0E
D4	5305 – 8989	3.684	1.34	25.5	7.55	13.22	17.67	2.75	14.92	1.0E
D1	0 – 1400	1.400	1.96	9.7	1.64	5.38	10.54	3.73	6.81	1.0E
D1	3477 – 5515	2.038	1.35	14.1	2.52	8.25	11.11	5.43	5.68	1.0E
D1	6677 – 8715	2.038	1.35	14.1	2.52	8.25	11.11	5.43	5.68	1.0E
D1	10792 – 12192	1.400	1.96	9.7	1.64	5.38	10.54	3.73	6.81	1.0E
D3	0 – 2314	2.314	1.19	16.0	2.16	7.48	8.87	-0.39	2.95	1.4W
D3	4391 – 7801	3.410	0.80	23.6	3.32	11.58	9.31	-0.57	3.24	1.4W
D3	9268 – 12192	2.924	0.94	20.3	2.75	9.50	8.91	-0.49	3.07	1.4W

HD = hold-down. Each segment is checked for 1.4W with 0.9D less the concurrent roof uplift as resisting load, and for 1.0E (NBC Table 4.1.3.2.-A, case 5) with 1.0D; the table shows the larger chord force and net hold-down tension and the case that produces it. Hold-down and base shear demands are to be used for the selection of hold-downs and anchors to the slab. Shear normal to the wall at the top / bottom track from wind on the wall: 1.63 kN/m. Shear wall deflection under wind: AISI S240 Eq. B5.2.5-1, limit h/500.

8.2 Seismic Drift

Table 30 – Seismic drift (NBC 4.1.8.13.(2)–(3), limit 0.025·hs)

Dir.	Wall	V segment (kN)	Segment w (m)	δ elastic (mm)	RdRo/IE	δ (mm)	Limit (mm)	Ratio
A	D2	13.22	3.684	3.632	1.00	3.632	123.1	0.029
A	D4	13.22	3.684	3.632	1.00	3.632	123.1	0.029
B	D1	8.25	2.038	3.443	1.00	3.443	68.6	0.050
B	D3	11.58	3.410	2.030	1.00	2.030	68.6	0.030

Elastic deflection of the most stressed segment by AISI S240 Eq. B5.2.5-1 under the segment's share of 1.0E, multiplied by RdRo/IE; single storey, so the storey drift equals the shear wall deflection at the top.

8.3 Roof Diaphragm

NBC 4.1.8.15.(1): diaphragms, chords, collectors and their connections shall be designed not to yield; the seismic diaphragm force is increased to the lateral load capacity of the SFRS (Sentence (1)(a)) by the factor k = Σ factored resistance of the lines / Σ seismic demand of the lines in that direction (not less than 1.0). The roof sheathing (OSB 11 mm) is BLOCKED at all panel edges.

Table 31 – Roof diaphragm (shear in kN/m; forces in kN; chord and collector = top track, Tr = 50.7 kN)

Dir	v 1.4W	k	v E·k	v des.	Ref. φvn	r	Screw bound	Chord force	r	Coll-ector	r	L/b	Lim.
A	1.65	1.93	5.58	5.58	7.21	0.77	34.9	17.02	0.34	9.92	0.20	1.33	4.0
B	0.66	1.92	4.41	4.41	7.21	0.61	34.9	10.09	0.20	22.17	0.44	0.75	4.0

v = diaphragm shear; k = NBC 4.1.8.15.(1)(a) amplification; r = ratio (demand / resistance); L/b = aspect ratio.

Conditional item. AISI S240 gives no tabulated diaphragm resistance for Canada. The resistance shown is the value of the United States table (B5.4.2.2-1) and is a reference only; the diaphragm resistance shall be confirmed for the Canadian design basis (test-based values for the sheathing and fastener schedule, or rational engineering analysis). The diaphragm shear check is therefore reported as CONDITIONAL in the summary. Diaphragm shear, chord and collector forces are the envelope of 1.4W and the amplified 1.0E.

8.4 Overall Stability

Uplift: 1.4W = 99.3 kN against 0.9D = 140.3 kN → ratio 0.71. Overturning (wind perpendicular to ridge): 340.6 kNm against 641.3 kNm → ratio 0.53. No net uplift on the building as a whole.

9 3D CHECK (SECOND ANALYSIS PROGRAM)

A three-dimensional linear model was built with a second analysis program from the same framing geometry, section properties and loads. Units N, mm. The truss part uses the same node, member and release definition as the 2D truss model, so for the trusses the comparison is a solver consistency check (Section 9.1), not an independent model; the wall framing (top track, cripple studs, headers, jack studs), the truss-to-wall load path and the lateral distribution are modelled independently of the design calculation. Out-of-plane degrees of freedom of the truss and wall planes are restrained (roof sheathing, wall sheathing); studs, jack and cripple studs are pinned at both ends; the truss pin releases and the rigid heel link are as in the 2D truss model. The gable ends are not part of the gravity runs; they are modelled separately for out-of-plane wind (full-height gable walls). Load combination for the gravity runs: 1.25D + 1.5S (balanced) + 1.0L.

9.1 Model Consistency

One truss on the same supports as the 2D design model (pinned / roller): the largest difference in member axial force is 0.000 % (member AL2), reaction 15.444 kN against 15.444 kN → consistent (tolerance 0.5 %).

9.2 Trusses Bearing on the Wall Framing

All 19 trusses bear on the top tracks of the eave walls through pin-ended links; top track → studs / cripple studs → box headers → jack studs. Equilibrium: Σ reactions 586.88 kN = Σ loads 586.88 kN. Truss reactions 15.44 … 15.44 kN against 15.44 kN in 2D (each truss is statically determinate on its supports, so the support flexibility does not change the truss forces).

Table 32 – Truss member axial forces, 3D vs 2D (kN; + tension)

Member	3D min	3D max	2D
UL1	-22.72	-22.72	-22.72
UR1	-20.08	-20.08	-20.08
UL2	-19.71	-19.71	-19.71
AL1	18.82	18.82	18.82
AL2	12.84	12.84	12.84
AL3	18.82	18.82	18.82
G1	-4.85	-4.85	-4.85
G2	6.53	6.53	6.53

Table 33 – Headers: 3D jack forces and moments against the 2D values with the same loads (kN, kNm)

Wall	x (mm)	Jacks left	Jacks right	2D jack	3D / 2D	3D M	Uniform-load M	3D / uniform	2D M (point loads)	3D / 2D
D1	1524	22.48	22.55	24.20	0.93	12.41	11.56	1.07	12.72	0.98
D1	5639	7.26	7.38	12.62	0.58	3.63	3.14	1.15	3.85	0.94
D1	8839	22.36	22.66	24.20	0.94	12.41	11.56	1.07	12.72	0.98
D3	2438	16.85	16.75	24.20	0.70	10.86	11.56	0.94	11.56	0.94
D3	7925	9.71	9.64	16.48	0.59	6.26	5.36	1.17	6.47	0.97

The 2D columns use the same loads as the 3D model (roof load only), so the ratios compare like with like. The design values in Table 15 and Section 7.3 are slightly larger: they also include the self-weight of the wall above the header.

Finding of the 3D check: the header moments exceed the equivalent uniform load moment ($qL^2/8$) by up to 17 % because the truss reactions reach the header as concentrated loads through the cripple studs. The design (Section 7.3) uses the concentrated-load moment; the 3D moments are within the design moments (3D / design ≤ 1). Jack forces in 3D are smaller than the design values.

9.3 Lateral Distribution with a Rigid Diaphragm

Table 34 – Shear wall line forces, rigid roof diaphragm, V = 44.04 kN with ±0.1·D accidental torsion

Wall	3D max (kN)	Case	Design line V (kN)	3D / design
D1	20.36	B, Mz +40.3 kNm	26.43	0.77
D2	24.56	A, Mz +53.7 kNm	26.43	0.93
D3	27.54	B, Mz -40.3 kNm	27.96	0.99
D4	24.56	A, Mz -53.7 kNm	26.43	0.93

Equilibrium in every case: the lines parallel to the load carry 44.04 kN, 44.04 kN, 44.04 kN, 44.04 kN = V. Finding of the 3D check: the stiffer eave wall D3 attracts more than the flexible-diaphragm share; the design (Section 8.1) uses the envelope of the flexible and rigid diaphragm assumptions, and all 3D line forces are within the design values.

9.4 Full-Height Gable Walls under Out-of-Plane Wind

Each full-height gable wall is modelled for 1.4W perpendicular to the wall: studs and double king studs (stiffness × number) pinned at the bottom track and laterally held at the rake track, the box header spanning between the king studs on its weak axis (pinned ends), the cripple studs from the header to the rake, the upper half of the window wind on the header and the lower half at the sill on the king studs. In-plane degrees of freedom are restrained.

Table 35 – Full-height gable walls, 3D against the design values (1.4W moments, kNm)

Wall	Member	3D	Design	3D / design
D2	king ×2 (s = 3838)	3.488	3.488	1.000
D2	king ×2 (s = 5305)	3.587	3.587	1.000
D2	header (weak axis)	0.666	0.946	0.703
D2	governing stud (s = 3205; all studs ≤ 1.000)	1.914	1.914	1.000
D4	king ×2 (s = 3838)	3.488	3.488	1.000
D4	king ×2 (s = 5305)	3.587	3.587	1.000
D4	header (weak axis)	0.666	0.946	0.703
D4	governing stud (s = 3205; all studs ≤ 1.000)	1.914	1.914	1.000

The king studs receive the window wind as point loads at the header and sill levels, and the cripple studs load the header in proportion to their position; the design uses these point loads (header reaction by the lever rule, window wind over the header span) rather than a uniform load over the tributary width

(design moment / uniform-load moment = 0.97–1.15), and the king moments agree with the 3D values. The header design moment (point loads lumped at mid-span) is conservative. Both models restrain the in-plane degrees of freedom and use the same loads, so this is a consistency check of the load path.

9.5 Shear Walls Modelled as Sheathed Panels

The lateral runs above use one secant spring per Type I segment. As a further check the shear walls were modelled physically in a separate lateral model (no gravity load): each Type I segment is a mesh of membrane shell elements (sheathing) between two chord studs (double stud, axial members), pinned at the base and tied to the rigid roof diaphragm at the top. The sheathing is orthotropic with negligible axial stiffness (panel joints and fastener slip) and a shear stiffness calibrated so that the segment reproduces the AISI S240 Eq. B5.2.5-1 (full-height gable wall segments: rectangles with the height of their longest stud, tied to the diaphragm level by auxiliary nodes) secant stiffness: $1/k = 2h^3/(3E \cdot Ac \cdot b^2)$ (chord term, produced by the chord members themselves because the shear flow accumulates linearly along the chord) + $h/(G \cdot t \cdot b)$ (sheathing shear and fastener slip). Single-segment calibration test (cantilever, unit load):

Table 36 – Shell shear wall calibration test

Wall	b (mm)	G·t (N/mm)	δ / (V/k)	Hold-down / (V·h/b)
D2	3684	5951	0.9984	1.0000
D1	1400	3090	0.9993	1.0000

Table 37 – Shear wall line forces: shell model against the spring model (kN)

Wall	Shell model	Case	Spring model	Shell / spring
D1	20.36	B, Mz +40.3 kNm	20.36	1.000
D2	24.56	A, Mz +53.7 kNm	24.56	1.000
D3	27.54	B, Mz -40.3 kNm	27.54	1.000
D4	24.56	A, Mz -53.7 kNm	24.56	1.000

Table 38 – Shear wall segments: shell model against the design (seismic, kN)

Wall	b (mm)	V shell	V design	Ratio	Hold-down shell	Hold-down design	Ratio
D1	1400	3.82	5.38	0.71	7.49	10.54	0.71
D1	2038	6.36	8.25	0.77	8.55	11.11	0.77
D1	1400	3.82	5.38	0.71	7.49	10.54	0.71
D1	2038	6.36	8.25	0.77	8.55	11.11	0.77
D2	3683	12.28	13.21	0.93	16.42	17.67	0.93
D2	3684	12.28	13.22	0.93	16.42	17.67	0.93
D3	2314	6.77	7.48	0.91	8.03	8.87	0.91
D3	2924	9.42	9.50	0.99	8.83	8.91	0.99
D3	3410	11.35	11.58	0.98	9.13	9.31	0.98
D4	3684	12.28	13.22	0.93	16.42	17.67	0.93
D4	3683	12.28	13.21	0.93	16.42	17.67	0.93

Findings: because the shell is calibrated to the same Eq. B5.2.5-1 stiffness as the springs, the equal line forces and the stiffness-proportional distribution within a line are consistency results, not independent evidence. The distribution by stiffness (wider segments take more than their length share) is covered in the design by taking, for every segment, the larger of the resistance share and the stiffness share (Section 8.1); the segment shears and hold-down forces of the shell model are within the design values. The new information of the shell model is the hold-down force, which equals V·h/b at the base.

An equivalent X-brace idealisation (two diagonals per segment, same calibrated stiffness) gives the same line forces but chord stud forces as low as 50 % of V·h/b, because the diagonals carry part of the overturning directly to the supports. It is therefore not used for chord stud or hold-down forces; the shell model is.

10 SUMMARY OF UTILISATION

Table 39 – Utilisation ratios (demand / factored resistance)

Group	Check	Ratio	Status	Note
Roof truss	top chord 550S162-97 — axial + bending	0.68	OK	UL1, 3b · S
Roof truss	top chord — bending + shear (H2-1)	0.35	OK	
Roof truss	top chord — concentrated load at panel point (E4.4.2.3)	0.34	OK	
Roof truss	top chord — concentrated roof load 1.3 kN (NBC 4.1.5.9)	0.30	OK	
Roof truss	bottom chord 550S162-68 — axial + bending	0.37	OK	AL3, 3b · S
Roof truss	bottom chord — bending + shear (H2-1)	0.40	OK	
Roof truss	web member 350S162-43 — compression (E4.4.4.1)	0.25	OK	
Roof truss	web member — tension	0.12	OK	
Roof truss	web member — end screws	0.73	OK	G1 4 / G2 4 / G3 4 / G4 4
Roof truss	heel — screw group	0.92	OK	44 × #10
Roof truss	heel — gusset plate (E4.5)	0.53	OK	
Roof truss	heel — bearing	0.82	OK	coped heel S240 E4.6.2.1(a): G2 shear × R, bearing stiffener 350S162-68
Roof truss	heel — shear	0.29	OK	
Roof truss	heel — uplift tie	0.33	OK	
Roof truss	ridge gusset plate (E4.5.1) + screws	0.97	OK	6 × #10 per chord
Roof truss	deflection S + L ≤ span/360	0.09	OK	2.3 mm
Wall studs	stud 600S162-43 — axial + bending (all-steel, bridging-braced)	0.79	OK	sheathing-braced, for information: 0.63
Wall studs	bridging 150U50-54 — accumulated brace force	0.69	OK	anchored every 6 studs
Wall studs	bridging — connection to stud	0.17	OK	
Wall studs	bridging — brace stiffness (S100 C2.3.1)	0.01	OK	
Wall studs	stud — wind deflection ≤ L/360	0.19	OK	
Wall studs	stud ↔ track web crippling (S100 G5)	0.61	OK	
Openings	king + jack ↔ track web crippling (S100 G5, two webs)	0.61	OK	
Gable end framing	full-height gable wall D2 — studs, axial + bending (L = 4618 mm)	0.92	OK	1.25D + 1.4W + 0.5S
Gable end framing	full-height gable wall D2 — wind deflection ≤ L/360	0.91	OK	
Gable end framing	full-height gable wall D2 — king stud group	0.85	OK	2 king studs
Gable end framing	full-height gable wall D2 — cripple studs above the header	0.38	OK	

Group	Check	Ratio	Status	Note
Gable end framing	full-height gable wall D2 — header out-of-plane (weak axis)	0.10	OK	
Gable end framing	full-height gable wall D2 — stud ↔ track web crippling (bottom end, S240 B3.2.5.1)	0.68	OK	
Gable end framing	full-height gable wall D2 — rake end clip screws	0.66	OK	2 × #10 per leg
Gable end framing	full-height gable wall D2 — rake track ↔ roof sheathing (steel side)	0.32	OK	
Gable end framing	full-height gable wall D2 — rake track bending between studs	0.60	OK	
Gable end framing	full-height gable wall D2 — header ↔ king connection (out of plane)	0.92	OK	2 × #10 per end
Gable end framing	full-height gable wall D2 — sill ↔ king connection + sill bending	0.31	OK	2 × #10 per end
Gable end framing	full-height gable wall D4 — studs, axial + bending (L = 4618 mm)	0.92	OK	1.25D + 1.4W + 0.5S
Gable end framing	full-height gable wall D4 — wind deflection ≤ L/360	0.91	OK	
Gable end framing	full-height gable wall D4 — king stud group	0.85	OK	2 king studs
Gable end framing	full-height gable wall D4 — cripple studs above the header	0.38	OK	
Gable end framing	full-height gable wall D4 — header out-of-plane (weak axis)	0.10	OK	
Gable end framing	full-height gable wall D4 — stud ↔ track web crippling (bottom end, S240 B3.2.5.1)	0.68	OK	
Gable end framing	full-height gable wall D4 — rake end clip screws	0.66	OK	2 × #10 per leg
Gable end framing	full-height gable wall D4 — rake track ↔ roof sheathing (steel side)	0.32	OK	
Gable end framing	full-height gable wall D4 — rake track bending between studs	0.60	OK	
Gable end framing	full-height gable wall D4 — header ↔ king connection (out of plane)	0.92	OK	2 × #10 per end
Gable end framing	full-height gable wall D4 — sill ↔ king connection + sill bending	0.31	OK	2 × #10 per end
Openings	header incl. end connection (governing: D1, x = 1524)	0.99	OK	
Openings	header deflection ≤ L/360 (governing: D1, x = 1524)	0.39	OK	
Openings	jack stud (governing: D1, x = 1524)	0.58	OK	jack studs required: 2
Openings	king stud (governing: D1, x = 1524)	0.98	OK	
Lateral system	shear wall D2 (direction A)	0.52	OK	

Group	Check	Ratio	Status	Note
Lateral system	shear wall D4 (direction A)	0.52	OK	
Lateral system	shear wall D1 (direction B)	0.58	OK	
Lateral system	shear wall D3 (direction B)	0.49	OK	
Lateral system	shear wall chord stud	0.83	OK	
Lateral system	shear wall deflection ≤ h/500	0.07	OK	
Lateral system	seismic drift ≤ 0.025·hs (4.1.8.13)	0.05	OK	
Lateral system	diaphragm, direction A — reference resistance (US table)	0.77	CONDITIONAL	v 5.58 kN/m
Lateral system	diaphragm, direction A — chord (top track tension)	0.34	OK	
Lateral system	diaphragm, direction A — collector (top track)	0.20	OK	
Lateral system	diaphragm, direction A — aspect ratio	0.33	OK	
Lateral system	diaphragm, direction B — reference resistance (US table)	0.61	CONDITIONAL	v 4.41 kN/m
Lateral system	diaphragm, direction B — chord (top track tension)	0.20	OK	
Lateral system	diaphragm, direction B — collector (top track)	0.44	OK	
Lateral system	diaphragm, direction B — aspect ratio	0.19	OK	
Lateral system	overall uplift (0.9D)	0.71	OK	
Lateral system	overall overturning	0.53	OK	

Overall result: **CONDITIONAL**. Highest utilisation 0.99 — header incl. end connection (governing: D1, x = 1524). All calculated ratios are within 1.0, but the following checks rest on provisions that do not cover the detail as built and are CONDITIONAL until confirmed before construction: diaphragm, direction A — reference resistance (US table); diaphragm, direction B — reference resistance (US table).

11 MATERIAL TAKE-OFF

Table 40 – Cold-formed steel quantities from the framing model

Section	Pieces	Total length (m)	Mass (kg)
150U50-54	15	54.5	35.7
350S162-43	76	164.7	278.8
550S162-68	19	173.7	578.4
550S162-97	38	210.3	974.2
600S162-43	125	329.2	745.1
600T125-43	15	93.5	179.8
800S162-68	14	22.4	94.9
Total	302	1048.4	2886.9

Plan area 111.5 m² → 25.9 kg/m² of cold-formed steel framing (walls incl. full-height gable walls and rake tracks, roof trusses; sheathing, fasteners, gusset plates, stiffeners and connectors are not included — accessories are listed in the spreadsheet). The cut list is issued as a separate spreadsheet.

12 WALL PANEL ELEVATIONS

Framing elevations of the four exterior wall panels, viewed from the exterior, generated from the framing model. The same drawings are issued in DXF format at full scale.

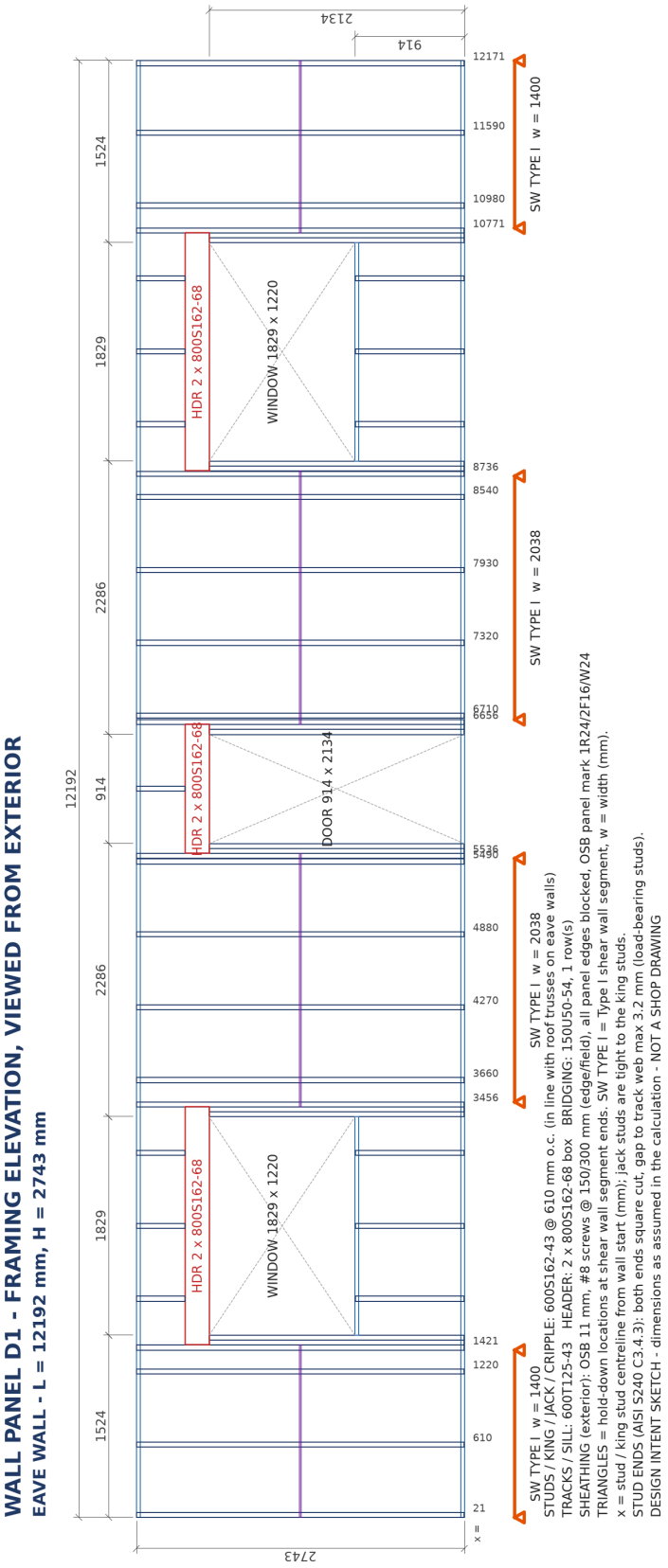


Figure 5 *– Wall panel D1 — framing elevation (eave wall, L = 12192 mm)*

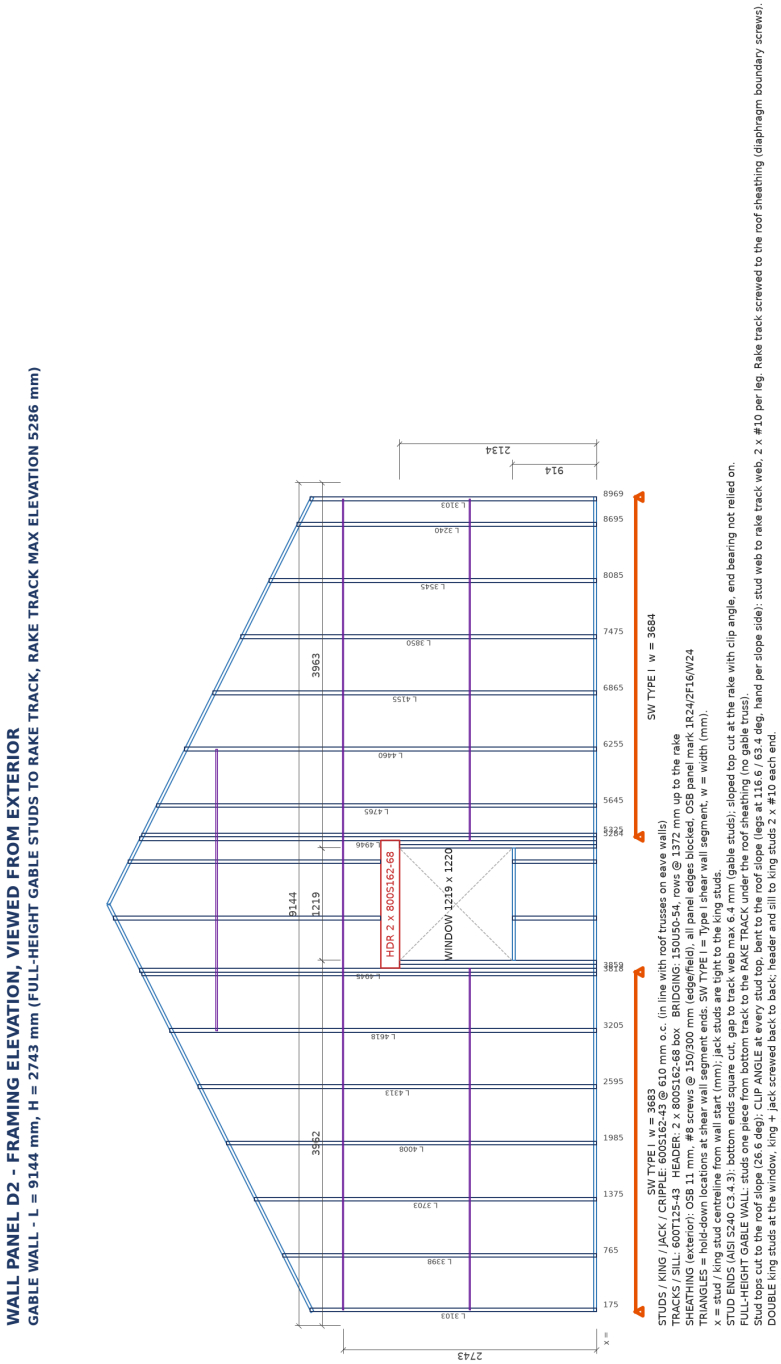


***Figure 6** – Wall panel D2 — framing elevation (gable wall, L = 9144 mm)*

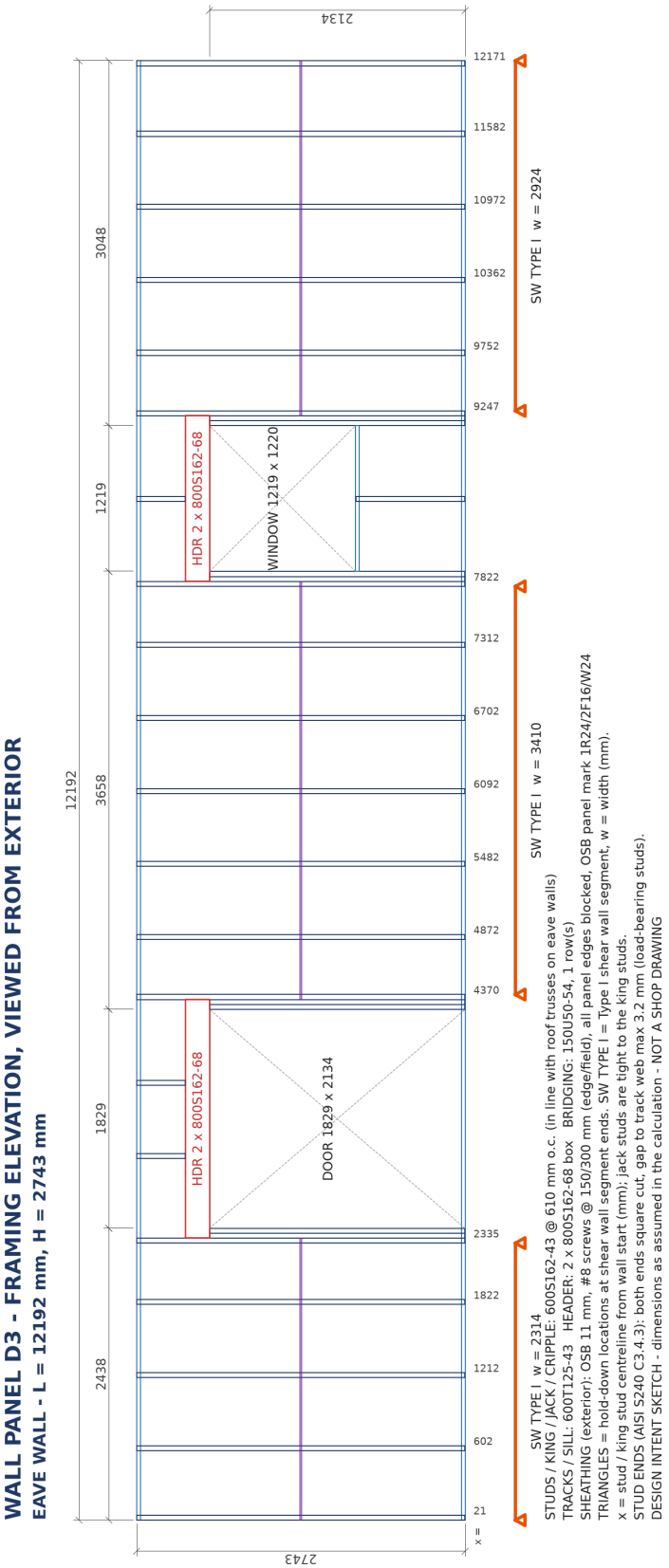


***Figure 7** – Wall panel D3 — framing elevation (eave wall, L = 12192 mm)*

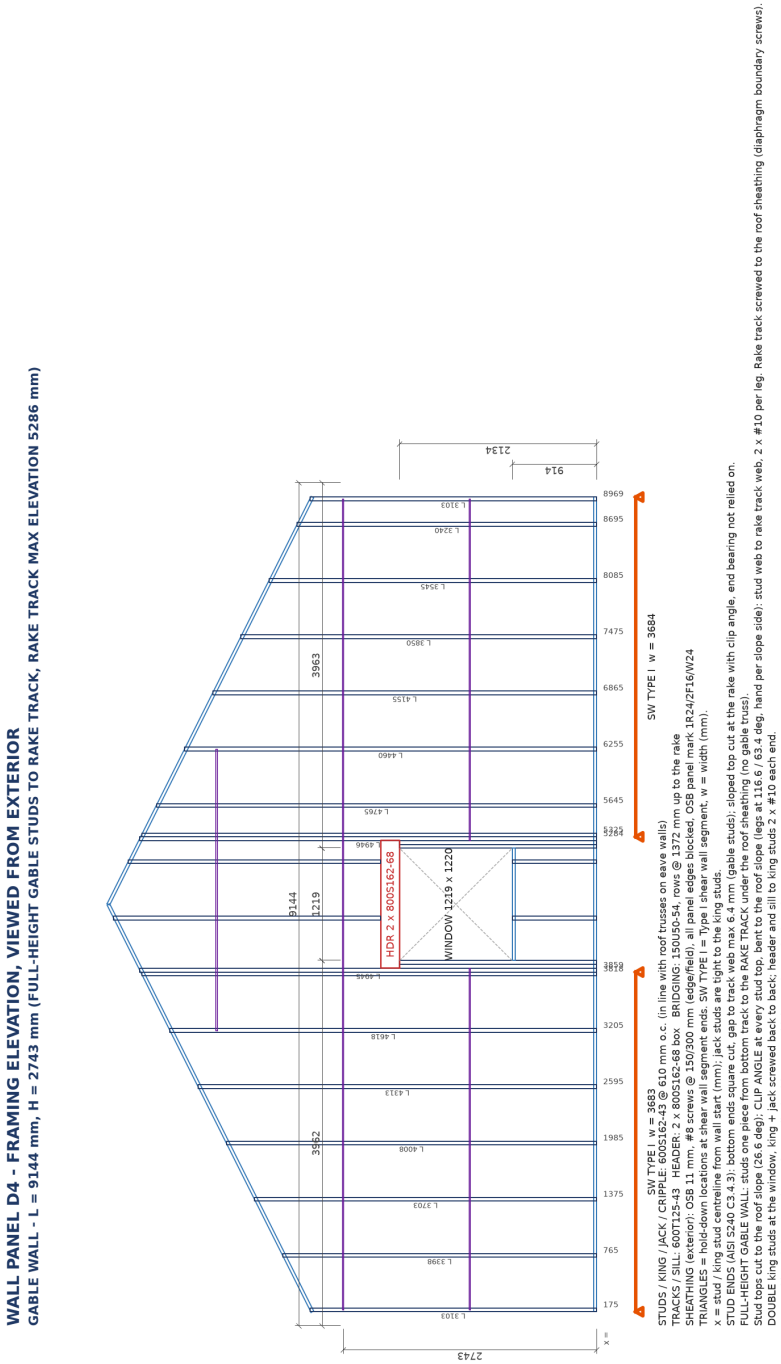


***Figure 8** – Wall panel D4 — framing elevation (gable wall, L = 9144 mm)*