CivE 495 Final Project

December 15, 2021

Contents

Final Project

About the design team

The paramedic station was designed by the Morning Wood Company. Engineers include:

- Thomas Charky (ID: 20791406)
- Benjamin Klassen (ID: 20761178)
- Ethan Truman (ID: 20767595)

Loading

Dead Loading

Roof Dead Loading (without joist self-weight)

- 0.25 kPa insulation and roofing
- 12mm drywall 0.1 kPa
- 0.1 kPa suspended
- 15.5mm sheathing 0.0753 kPa pg.972 of WDM (linear interpolated)

TOTAL= 0.5253 kPa +SW (assume 19% moisture content)

Snow Loading

Formulas for snow load were given, with variables from location tables required to solve formula. We found the Sr and Ss value to be 0.4 and 1.3 from the location table for Tilsonburg and we calculated the snow loading.

Snow Loading is 1.8 kPa ULS, 1.296 kPa SLS (Sr 0.4, Ss 1.3)

Wind Loading

(1/50 using 0.44 kPa (q), Iw=1.25(ULS) 0.75(SLS), z=1,y=6 CpCg=0.75 side, 1.15 corner - refer to table 4.1.7.6)

The 1/50 value of 0.44 kPa was found in the tables in the NBC for Tilsonburg Ontario. Iw values are importance factors. 1.25 for ULS and 0.75 for SLS. CpCg was found in table 4.1.7.6 to be 0.75 for the side and 1.15 for the corner.

Wind Loading for Design of Singular Stud

- CpCg=-1.8, CgiCpi= -0.9

P(external ULS) = -0.891 P(internal ULS) =-0.4455

P(External SLS) = -0.5346 P(internal SLS) =-0.2673

For the CpCg values found in this question, the worst cases were taken into consideration from the PDF slides provided by Professor Zurell. Interior pressure was taken into account because we only considered a single wall, not the full system.

Wind Loading for Shearwall ULS: Side: 0.37125 (1 side), 0.27225 (4 side)

ULS: Corner: 0.56925 (1E), 0.396 (4E)

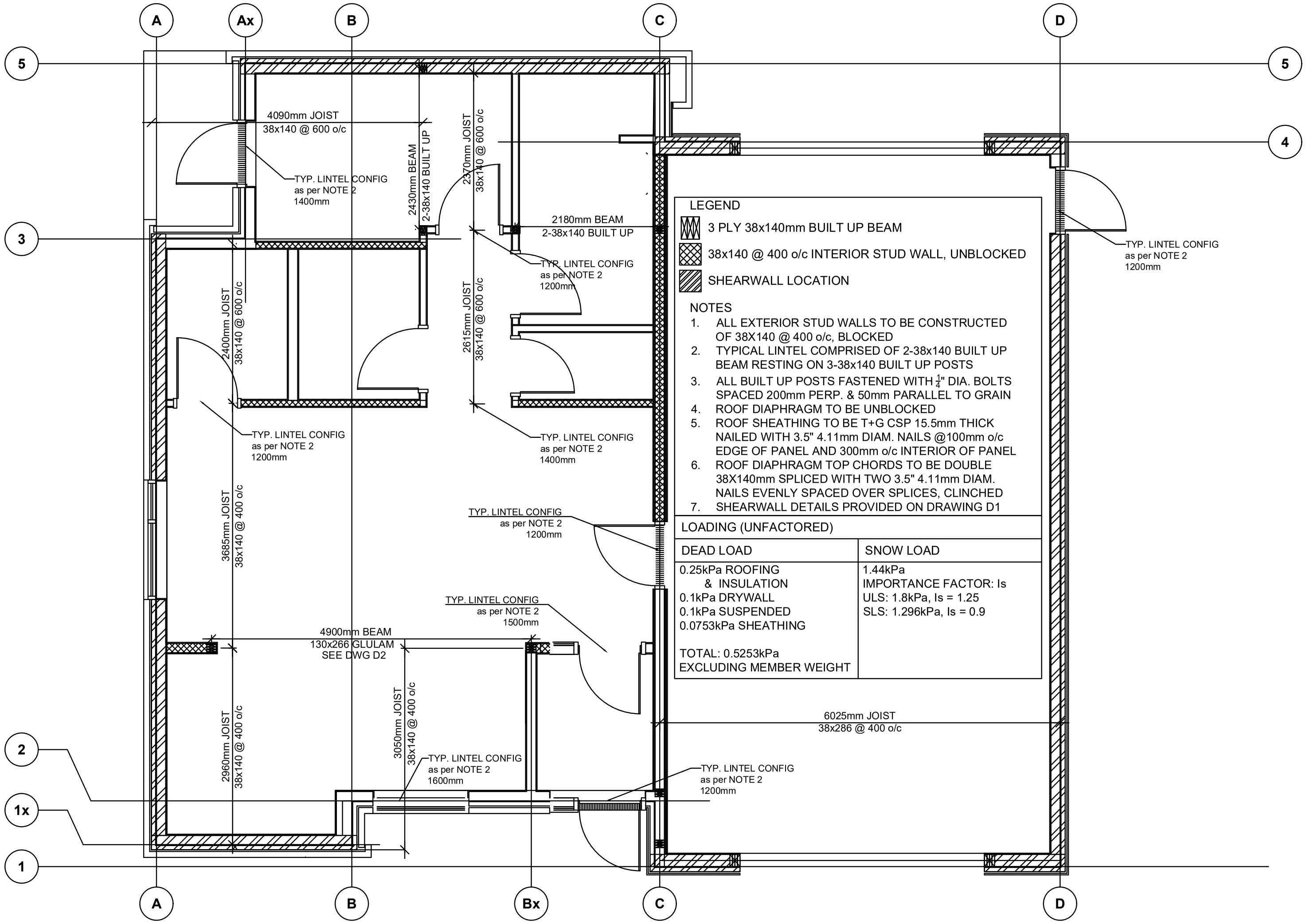
SLS: Side: 0.22275 (1 side), 0.16335 (4 side)

SLS: Corner: 0.34155 (1E), 0.2376 (4E)

Shear wall values ignored the interior pressure because it cancels itself out.

General Framing Plan

See below for the general framing plan and foundation plan:



LEGEND

- 3 PLY 38x140mm BUILT UP BEAM
- 38x140 @ 400 o/c INTERIOR STUD WALL, UNBLOCKED
- SHEARWALL LOCATION

NOTES

1. ALL EXTERIOR STUD WALLS TO BE CONSTRUCTED OF 38X140 @ 400 o/c, BLOCKED
2. TYPICAL LINTEL COMPRISED OF 2-38x140 BUILT UP BEAM RESTING ON 3-38x140 BUILT UP POSTS
3. ALL BUILT UP POSTS FASTENED WITH $\frac{1}{4}$" DIA. BOLTS SPACED 200mm PERP. & 50mm PARALLEL TO GRAIN
4. ROOF DIAPHRAGM TO BE UNBLOCKED
5. ROOF SHEATHING TO BE T+G CSP 15.5mm THICK NAILED WITH 3.5" 4.11mm DIAM. NAILS @100mm o/c EDGE OF PANEL AND 300mm o/c INTERIOR OF PANEL
6. ROOF DIAPHRAGM TOP CHORDS TO BE DOUBLE 38X140mm SPLICED WITH TWO 3.5" 4.11mm DIAM. NAILS EVENLY SPACED OVER SPLICES, CLINCHED
7. SHEARWALL DETAILS PROVIDED ON DRAWING D1

LOADING (UNFACTORED)

DEAD LOAD	SNOW LOAD
0.25kPa ROOFING & INSULATION 0.1kPa DRYWALL 0.1kPa SUSPENDED 0.0753kPa SHEATHING TOTAL: 0.5253kPa EXCLUDING MEMBER WEIGHT	1.44kPa IMPORTANCE FACTOR: Is ULS: 1.8kPa, Is = 1.25 SLS: 1.296kPa, Is = 0.9

REVISIONS	DATE
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By:
MORNING WOOD Engineering
District Office

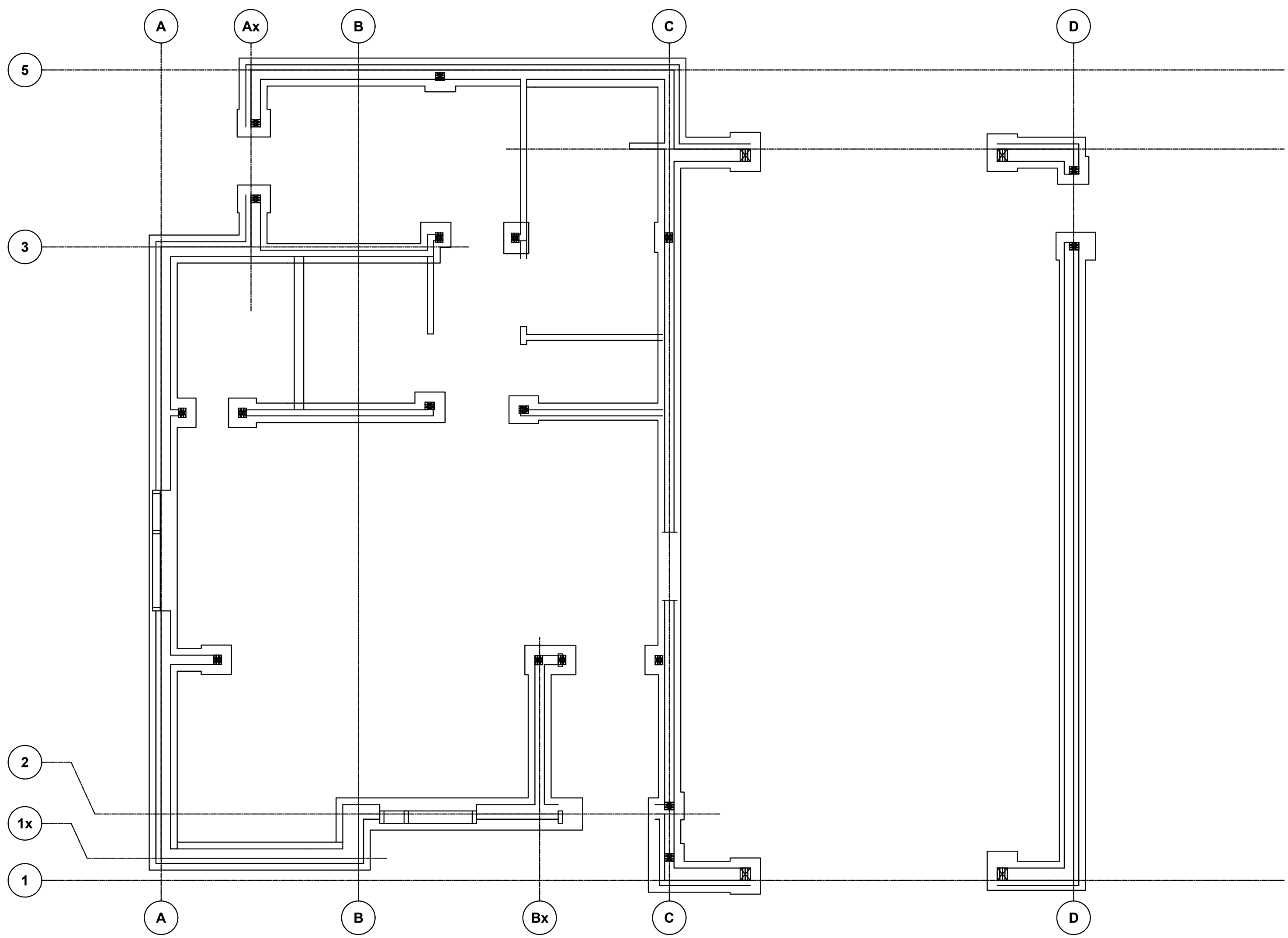
PROJECT:
Paramedic Stn. Tillsonburg ON

TITLE:
Framing Plan

DATE:
December 15, 2021

SCALE: NTS

DRAWING: S1



REVISIONS	DATE
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Notes:

FOUNDATION TYPICALLY 2FT WIDE ALONG LOAD-BEARING WALLS. 3FT WIDTH USED FOR ISOLATED FOOTINGS AT COLUMN LOCATIONS. TO BE CONFIRMED BY GEOTECHNICAL ENGINEER

By:
MORNING WOOD
Engineering
District Office

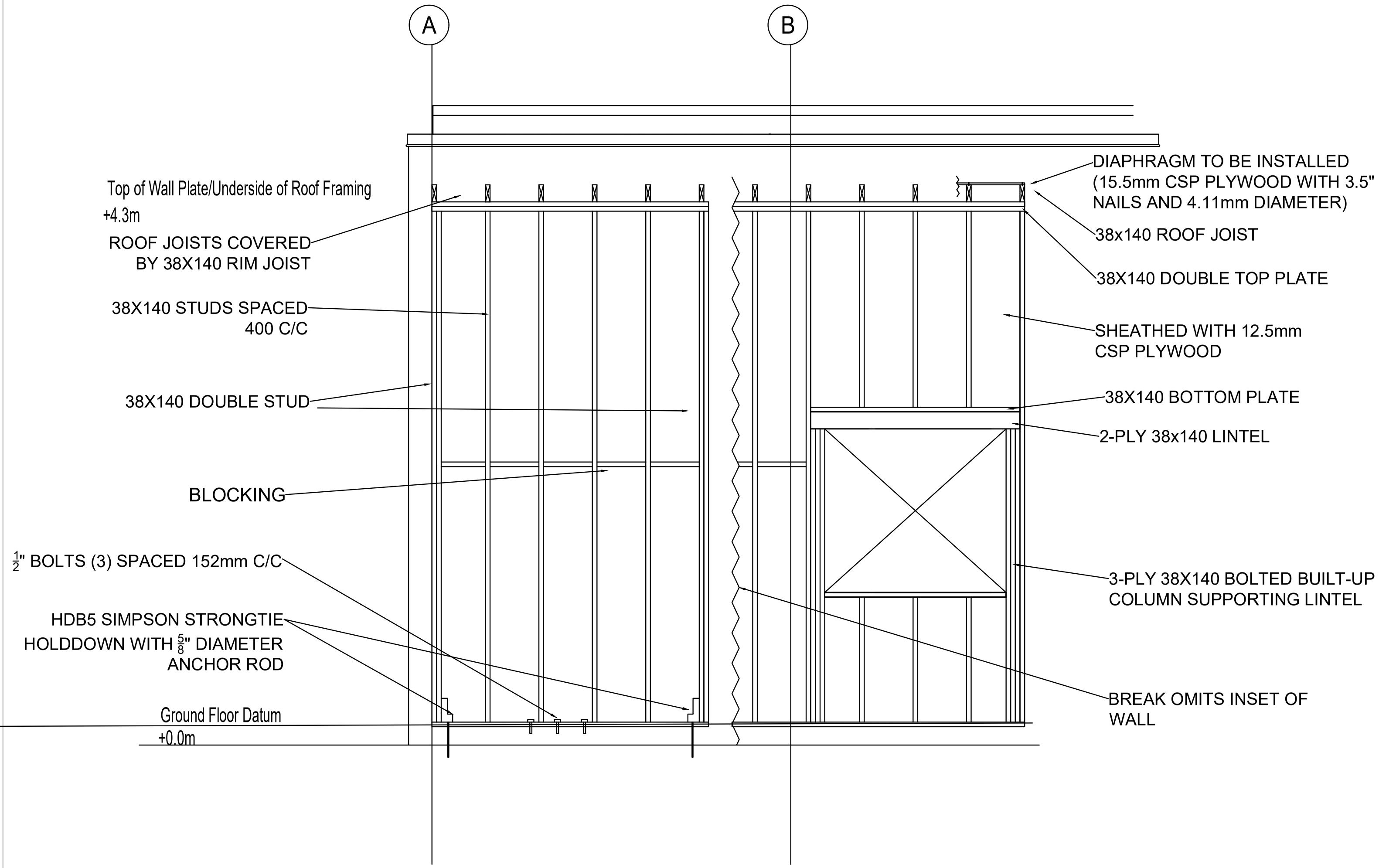
PROJECT:
Paramedic Stn. Tillsonburg ON

TITLE:
FOUNDATION PLAN

DATE:
December 15, 2021

SCALE:
NTS

DRAWING:
S2



REVISIONS	DATE
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Notes:

BOTH SIDES OF SHEARWALL ARE SHEATHED

SHEATHING IS NAILED TO OUTSIDE STUDS AT 150mm C/C AND TO THE INSIDE STUDS AT 300mm C/C. 3.5" NAILS WITH 4.11mm DIAMETER REQUIRED

By:
MORNING WOOD Engineering
District Office

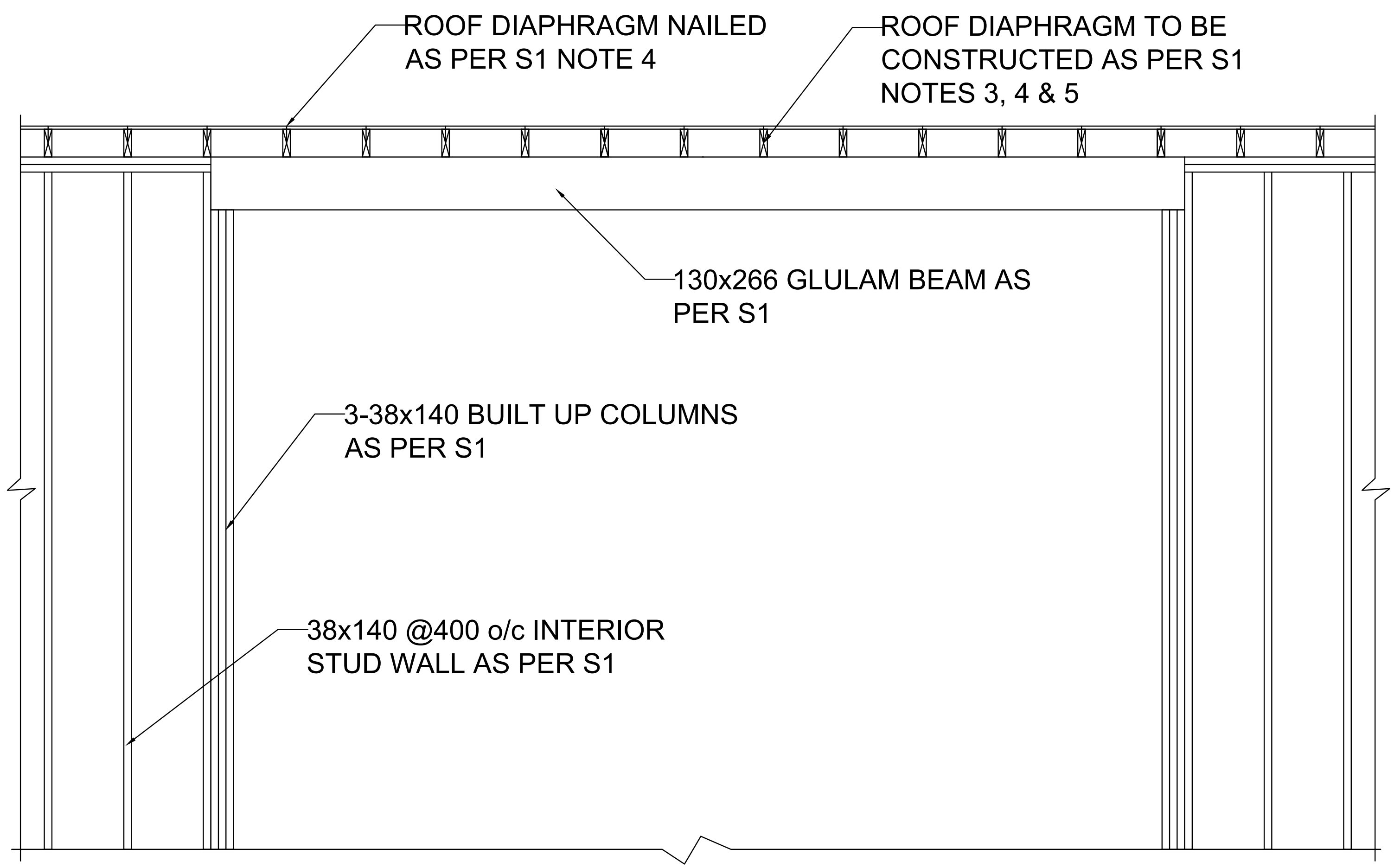
PROJECT:
Paramedic Stn. Tillsonburg ON

TITLE:
FRONT SHEARWALL/WALL OPENING

DATE:
December 15, 2021

SCALE: NTS

DRAWING: D1



REVISIONS	DATE
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By:
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District Office

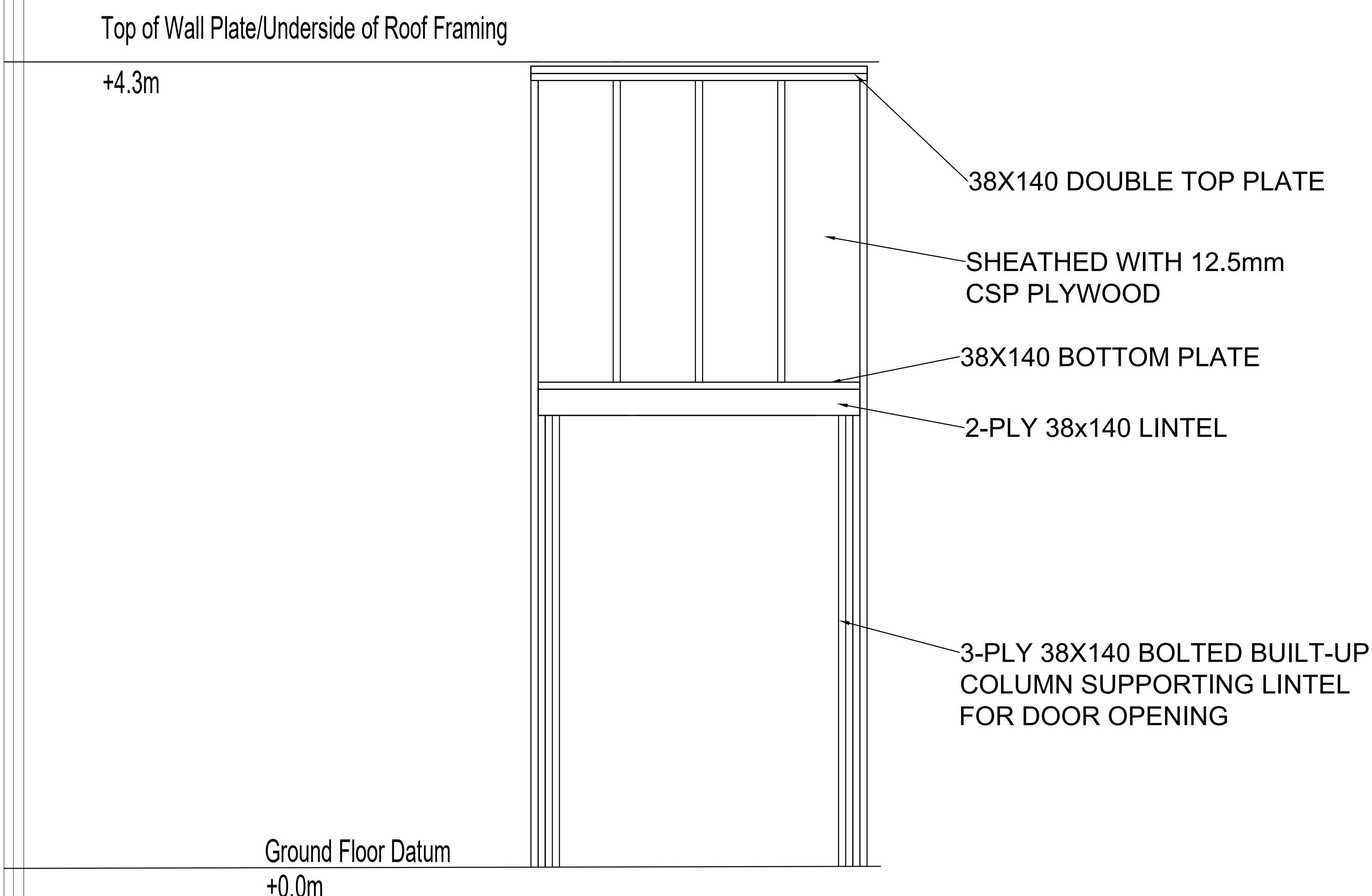
PROJECT:
Paramedic Stn. Tillsonburg ON

TITLE:
Glulam Beam Over Kitchen

DATE:
December 15, 2021

SCALE: NTS

DRAWING: D2



REVISIONS	DATE
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Notes:

SHEATHING IS NAILED TO OUTSIDE STUDS AT 150mm C/C AND TO THE INSIDE STUDS AT 300mm C/C. 3.5" NAILS WITH 4.11mm DIAMETER REQUIRED

By:
MORNING WOOD Engineering
District Office

PROJECT:
Paramedic Stn. Tillsonburg ON

TITLE:
DOOR OPENING

DATE:
December 15, 2021

SCALE: NTS

DRAWING: D3

Joist, Beam, Stud Wall, and Lintel Design

The joists, beams, stud wall, and lintel were all vital components of the general framing plan.

See the attached for joist designs, including joists that:

- Spanned over the garage
- Spanned across the cantilever
- Spanned across the worst case scenario

Stud Wall

Length = 4300 - 2(38)
= 4224 mm

W_f = 0.891 + 0.4450
↑ external ↑ internal
= 1.3365 kPa

DL_f = 0.5253 (roof load) × 3.013 (half of length of garage) + Self weight of top plate + Self weight of joists in garage

↳ Joists in garage = 4.6 kN/m³ × 0.038 · 0.286 · 3.013 = 0.1506 kN / Joist.

Joists spaced @ 400 c/c. ∴ approximated as UDL

∴ 27 joists fit across the garage. (Dim of 10910 mm length for the studwall from line ④ → line ①)

∴ UDL = $\frac{27 \cdot 0.1506}{10.91}$ = 0.373 kN/m

Self weight of top plate (use 2 2×6) = 4.6 kN/m³ × 0.038 × 0.140 · 2
= 0.049 kN/m

Total DL roof = 0.373 + 1.583 + 0.049 = 2.0 kN/m

DL_f = 1.25 (2) = 2.50 kN/m

SL_{ULS} = 1.8 kPa × 3.013 = 5.4234 kN/m

SL_f = 1.5 (5.4234) = 8.1351 kN/m , $S_{Lf(0.5)}$ = 0.5 (5.4234) = 2.7117

3 load cases:

1. 1.25D + 1.5S
2. 1.25D + 1.5S + 0.4W
3. 1.25D + 1.4W + 0.5S

Try 2x6 @ 406 c/c

Case 1

DL = (2.5)(0.4)

= 1 kN

SL = 8.1351 · 0.4

= 4.06755 kN

P_f = DL + SL = 5.0671 kN

From WDM, using 4.4 m piece w/ $K_D = 1$, $K_H = 1.1$, $P_r = 20.5$ kN

∴ $P_r >> P_f$, OK

Case 2

DL + SL = 5.067

0.4 · 0.4 · 1.3365 = 0.21384

From WDM, using 5m 2x6, w/ $K_D = 1.15$,

$P'_r = 9.71$, $w'_r = 0.344$

Since $P'_r > P_f$, $w'_r > w_f$, OK

Case 3

DL = ~~1 kN~~ kN

SL = 2.717 · 0.5

= 1.35585 kN

P_f = DL + SL

= 2.36 kN

w_f = 1.4 · 0.4 · 1.3365

= 0.74844

Using WDM, at 5m, $P'_r = 3.24$ and $w'_r = 0.78$.

∴ $P'_r > P_f$ and $w'_r > w_f$, passes

Joist Design - Over garage

$DL = 0.5253 \cdot T_w + S_w$

Factored $DL = 1.25 DL = 0.657\ T_w + 1.25\ S_w$, $S_w = 4.6\ kN/m^3 \times 0.038 \times d$
$S_{wf} = 1.25 S_w = 0.2185 d$

SL factored $= 1.8 \cdot 1.5 \cdot T_w = 2.7 T_w$

SL unfactored $= 1.296\ T_w$

$L = 5816\ mm$

a) Try 2x10 @ 400 c/c

Factored Loads

$w_D = 0.657 \cdot 0.4 + 0.2185 \cdot 0.235$
$= 0.3141\ kN/m$

$w_L = 2.7 \cdot 0.4$
$= 1.08\ kN/m$

$\therefore w_f = 1.3941$

$M_f = \frac{1.3941 \times 5.816^2}{8} = 5.8945\ kN \cdot m > 5.7\ kN \cdot m$ (Case 2 from WDM).

$\therefore$ fails in moment

b) Try 2x12 @ 500 c/c

Factored Loads

$w_D = 0.657 \cdot 0.5 + 0.2185 \cdot 0.286$
$= 0.391\ kN/m$

$w_L = 2.7 \times 0.5$
$= 1.35\ kN/m$

$\therefore w_f = 1.741\ kN/m$

$M_f = \frac{1.741 \times 5.816^2}{8} = 7.36\ kN \cdot m < 7.70$ (Case 2 WDM). $\therefore$ Passes Moment

$V_f = \frac{1.741 \times 5.816}{2} = 5.06\ kN < 13.7\ kN$ (Case 2 WDM). $\therefore$ Passes Shear

$DL_{SLS} = 0.5253 \cdot 0.5 + 4.6 \times 0.038 \times 0.286 = 0.313$ kN/m

$SL_{SLS} = 1.296 \cdot 0.5 = 0.648$

$w = 0.961$ kN/m

For deflection requirements,

Total Load $L/180 = 32.31 > \dfrac{5(0.961)(5816)^4}{384EI}$, $EI_{req} = 443E9$ Nmm

Transient Load $L/240 = 24.23 > \dfrac{5(0.648)(5816)^4}{384\,EI}$, $EI_{req} = 399E9$ N·mm

$EI = 704E9$ N·mm, ∴ passes deflection

∴ 500 c/c works. However, 400 c/c used to accomodate standard sheathing dimensions.

Joist Design - Span over washrooms

$L = 2523$ mm

a) Try 2x6 @ 600 c/c

$DL_f = 0.657 \cdot 0.6 + 0.2185 \cdot 0.140 = 0.425$ kN/m $\quad DL = 0.5253 \cdot 0.4 + 4.6 \times 0.038 \times 0.140 = 0.2257$

$SL_f = 2.7 \cdot 0.6 = 1.62$ kN/m $\quad SL = 1.296 \cdot 0.4 = 0.5184$

∴ $w_f = 1.3623$ kN/m $\quad w_s = 0.744$

$M_f = \dfrac{1.3623 \cdot 2.523^2}{8} = 1.084 < 1.27$ kN·m (Case 2 WDM), ∴ passes moment

$V_f = \dfrac{1.3623 \cdot 2.523}{2} = 1.719 < 7.24$ kN·m (Case 2 WDM), ∴ passes shear

Total load deflection worst case, $L/180 = 14.017 > \dfrac{5(0.744)(2523)^4}{384\,EI}$

$EI_{req} = 28E9$ N·mm, ∴ passes EI

Largest Length Joist (Length = 3685 mm)

Try 2x6 @ 400 c/c

ULS

$SL = 1.8 \times 1.5 \times 0.4$
$= 1.08$ kN/m

$DL = [1.25(0.5253) \times 0.4 + 4.6 \times 0.038 \cdot 0.140 \times 1.25$
$= 0.2932$ kN/m

Total Load = 1.3732 kN/m

SLS

$SL = 1.296 \times 0.4$
$= 0.5184$ kN/m

$DL = 0.5253 \times 0.4 + 4.6 \times 0.038 \cdot 0.14$
$= 0.2346$

TL = 0.753 kN/m

$M_f = \frac{1.3732 \cdot 3.685^2}{8} = 2.330$ kN·m $< M_r$ (WDM pg. 43), ok

$V_f = \frac{1.3732 \cdot 3.685}{2} = 2.53$ kN $< V_r$ (WDM pg. 43), ok

$L/240$ (transient), transient governs
$L/180$ (total)

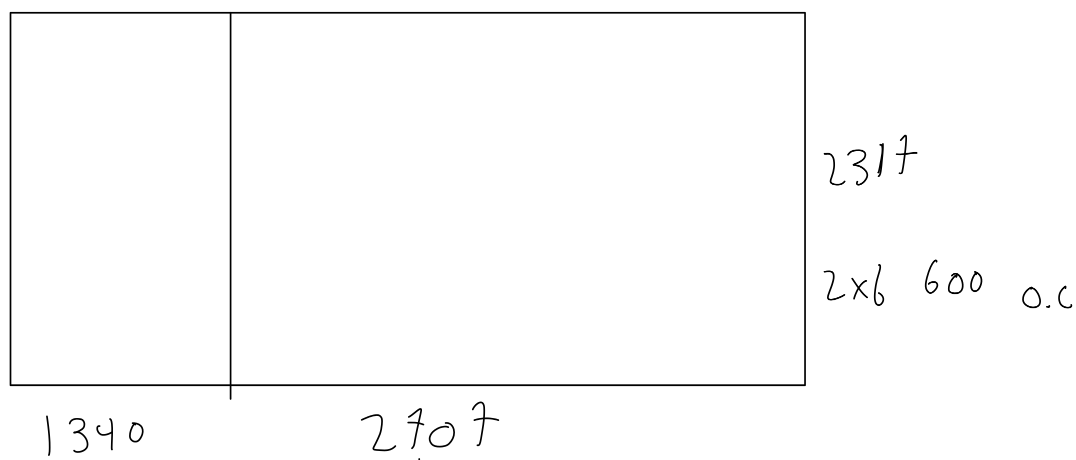
$\frac{3685}{240} = 15.35$ mm

$$15.35 = \frac{5 \cdot 0.5184 \cdot 3685^4}{384 EI}$$

$EI_{req} = 81E9$ N·mm² < WDM Value (95.6 N·mm²)

∴ 2x6 @ 400 c/c

Northwest cantilever Calculations



ULS: $1.25(DL) + 1.5(LL)$	SLS: $DL + LL$
$= 1.25(0.5253 + 4.6 \cdot 0.038 \cdot 0.140) + 1.5(1.8)$	$=(0.5253 + 4.6 \cdot 0.038 \cdot 0.140) + 1.296$
$= 3.387215$ kN/m	$= 1.845772$ kN/m 1m spacing
	$=(0.5253 \times 0.6 + 4.6 \cdot 0.038 \cdot 0.140) + 1.296 \cdot 0.6$
	$= 1.117252$ kN/m 0.6m spacing

Calculate factored loading

$$V_3 = \frac{3.387}{2 \cdot 2.71}\left(2.71^2 + 1.34^2\right)$$

$V_f = 5.711469$ kN at support intermediate

Moment

$$M_1 = \frac{3.387}{8 \cdot 2.71^2}(2.71 + 1.34)^2(2.71 - 1.34)^2$$

$= 1.7721$ kN·m

$$M_2 = \frac{3.387 \cdot 1.34^2}{2} = 3.0408 \text{ kN·m}$$

With large spacing
$M_r =$

$M_r = 3.61$ kN·m
$V_r = 9.38$ kN

Passes with 1m spacing

allowable deflection

$\frac{L}{240} \Rightarrow$ Transient $= 11.3$ mm

$\frac{L}{180} \rightarrow$ Total $= 15.04$ mm

$\Delta_x = \frac{wx}{24EIL}\left(L^4 - 2L^2x^2 + Lx^3 - 2a^2L^2 + 2a^2x^2\right)$
between supports

18.67

$= 3.9$ mm ✓ 2x6 Passes

2x4 Does not pass!

$\Delta_x = \frac{wx}{24EI}\left(4a^2L - L^3 + 6a^2x_1 + x_1^3\right)$
Overhang

11.37

$= 4.465$ mm ✓ 2x6 Passes

$\therefore$ Passes for all deflection cases for 2x6 at 600 O.C

Design Beam Supporting cantilever Section

Beam is 2.3m long

$= 2532.3 \text{ mm}/2$

$= 1266.1$ mm

ULS: $1.25(0.665 + 0.0539) + 1.5(2.28)$

$= 4.32$ kN/m

SLS: no worries :)

$M_f = \frac{wL^2}{8} = 2.8566$ kN

$V_f = 2.16$ kN

What can support this?

2-ply No.1/No.2 SPF

38×140

$M_R = 4.06$ kN/m

$V_R = 14.7$ kN

*4 Joists for dL

$DL_{sw} = 4(38 \times 140 \times 1266.1) \times 4.6$

$= 0.124$ kN

$= 0.0539$ kN/m

$DL_A = 0.5253$ kpa $\cdot$ 1.266

$= 0.665$ kN/m

$LL = 1.8$ kpa $\cdot$ 1.266

$= 2.28$ kN/m

Post supporting south end of the above Beam

$= (4.32 \times 2.3)/2$

$= 4.968\ \text{kN} \downarrow$

Since 3-ply 38x140 is shown to provide resistance in great excess of this load in the next section, it can be used as a column.

See the attached for beam, column, and opening designs and drawings, including:

- Large glulam beam
- Built-up beam
- Column supporting beams (and lintels)
- Door opening (design also supports lintel since it is the worst case)

Design - Beam Calculations - largest Span - 4640 -

• assume 12 joists attach total

2x6 - Joists attach 400 O.C.

ULS: $1.25(DL) + 1.5(LL)$

$= 1.25(0.98252) + 1.5(1.377)$

$= 3.29365$ kN/m

SLS: 2.36 kN/m

DL w/o SW: 0.5253 kPa

SW: $\underset{\text{No. Joists}}{12}\,(\underbrace{38 \times 140}_{\text{2x6 wood}} \times \underset{\text{half of Span}}{1877.5}) \cdot \gamma_{wood}$ — 4.6 kN/m³

$= 0.55135$

Total DL: 0.1188 + Applied + built up beam

Total LL: 1.8 kPa

$M_f = \frac{wL^2}{8} = \frac{3.29 \cdot 4.64^2}{8}$

$= 8.85$ kNm

$V_f = 7.64$ kN

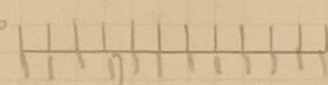
Tributary Area: $\underset{\text{length of Beam}}{4640} \times \underset{\text{half of Span}}{1877.5}$

$= 8711600$ mm²

$M_R = 9.02$ kN·m
$V_r = 24.9$ kN
] WDM pg. 90 | SPF 1/2 |

$E_s I = 562 \times 10^9$ N·mm²

Joist →



max deflection: $\frac{L}{180}$ for total load

$\Delta_{max} = \frac{5wL^4}{384\,EI} = 1.24$ mm

<

$\Delta_{max\ allow} = \frac{4640}{180} = 25.78$ mm

3-ply 2x8 BUB

SW: $3(38 \times 184) \times 4.6$ kN/m³

$= 0.0964596$ kN/m

Can conclude that built up beam will not suffice for this portion, moving up to glulam

Total DL: $\underset{\text{SW Joists}}{0.19} + \underset{\text{SW Beam}}{0.0964} + \underset{\text{applied}}{1.71}$

$= 1.92$ kN/m

Total LL: 1.8 kPa

= kN/m

Trib width on both sides

need Safety factor

Design – Beam Calclations – 4610 mm Span

Check K_{zbg}

$\updownarrow$

K_L

✓ governs

$$K_{zbg} = \left(\frac{130}{130}\right)^{1/10}\left(\frac{610}{266}\right)^{1/10}\left(\frac{9100}{4616}\right)^{1/10} \leq 1.3$$

$$= \boxed{1.16 \leq 1.3}$$

$K_L = ?$

$= 1.0$

~~given this factor there is a chance a the smaller could work~~

ULS: $1.25DL + 1.5LL$

$= 1.25(1.694 + 0.205 + 0.159) + 1.5(5.814)$

$= 11.2935$ kN/m

SLS: 7.872 kN/m

$$M_f = \frac{11.29 \cdot 4.61^2}{8} = 30 \text{ kNm}$$

$V_f = 26$ kN

130x266 glulam Values

$M'_r = 35.3$ kNm

$V_r = 36.3$ kN

$$\Delta = \frac{5wL^2}{384EI} = 1 \text{ mm}$$

good!

• glulam 130x266

$L_e = 1.92 \cdot 400$

$= 768$

$$C_B = \sqrt{\frac{L_e d}{b^2}}$$

$$= \sqrt{\frac{768 \cdot 266}{130^2}}$$

$= 3.48 < 10$

LL: 1.8×3.23

$= 5.814$

Trib Area

$= 3660/2 + 2790/2$

$= 3225$ mm $\updownarrow$

$\leftrightarrow$ 4616 mm

0.5253 kPa $\cdot$ 3.225 m

w/o SW $= 1.694$ kN/m

Joist weight

$= 12 \cdot (38 \times 140 \times (1830 + 1395)) \cdot 4.6$

$= 0.947$ (total kN)

$= 0.205$ kN/m

glulam SW

$= 130 \times 266 \cdot 4.6$

$= 0.159$ kN/m

Worst case column Scenario calculations

$= 11.29\ kN/m \cdot 4.64\ m$

$= 26.1928\ kN$
(for 1 post)

From WDM: SPF No.2
$P_r = 60.2$ (3.5m)
$P_r = 50.2$ (4.0m)
144 x 144 mm

way too high

lets try built up 3-ply

SPF No.1/No.2 38x140 built up column

$P_r = 32.3$ kN for 3.8m or

28.9 for 4.0m

Header beam above Doorways

ULS: $1.25(DL) + 1.5(LL)$

$= 8.265 \text{ kN/m}$

SLS: 5.795 kN/m

$$M_f = \frac{wL^2}{8} = 0.9324 \text{ kNm}$$

$V_f = 4.133 \text{ kN}$

WDM:
2-ply 38 X 140 SPF 1/2

$M_r = 4.06$ $V_r = 14.7$

Check deflection

$$= \frac{5wL^2}{384EI}$$

$= 0.267 \text{mm}$ (no problem)

Using 2-ply 2x6

Span over door 950mm (conservative)

2 Joists (for weight)

$= 1807.5 + 1159$

$= 2966.5$

$= 2 \cdot (38 \times 140 \times 2966.5) \cdot 4.6$

$= 0.14519 \text{ kN} / 0.950 \text{ m}$

$= 0.152834 \text{ kN/m}$

$= 0.5253 \cdot 2.966$

$= 1.5583 \text{ kN/m}$

Live load

$= 4.084182 \text{ kN/m}$

Column supporting Header load (around doors)

$= 8.265 \cdot 0.950$

$= 7.85175 / 2$

$= 3.925875 \text{ kN} \downarrow$

lets try built up

SPF No.1/No.2 38x89 built up column

3.8m tall $\rightarrow P_r = 5.23$ kN $\rightarrow$ 2 ply

adjust to 2-ply $\rightarrow$ 2x6

$P_r = 8.23$ kN for 3.8m

Checking 3-ply SPF 2x6

$L_b = 4200$ mm

$C_c = \frac{L_b}{114} = 36.84 < 50$, OK

$K_{zcb} = 6.3/(d_b \cdot L_b)^{0.13} < 1.3$ OK

$= 1.15$

$C_c = 36.84$

$$K_{cb} = \left[1 + \frac{f_c K_{zc} C_c^3}{35 E_{05} K_{SE} K_T}\right]^{-1} = \left(1 + \frac{11.5 \cdot (1.15)(36.84)^3}{35 \cdot 6500 \cdot 1 \cdot 1}\right)^{-1}$$

$= 0.256$

$P_r = \max\{K P_{r1} \text{ or } P_{r2}\}$

$\phi = 0.8$, $f_c = 11.5$ MPa, $K_D = 1.0$

no load sharing $K = 1.0$

$K_{SC} = K_{SE} = 1.0$

$K_T = 1.0$

$E = 9500$ MPa

$E_{05} = 6500$ MPa

$P_{rb} = \phi f_c A K_{zcb} K_{cb} K$

$= 0.8 \cdot 11.5 \cdot 115 \cdot 140 \cdot 1.15 \cdot 0.256 \cdot 0.75 \times 10^{-3}$

$\boxed{= 32.704 \text{ kN}}$ — resistance of 3-ply 2 by 6

*We will use this for safety

Checking 2ply-2 x 6 SPF

$\phi = 0.8$, $f_c = 11.5$, $K_D = 1.0$, $K_{SE} = 1.0$, $K_T = 1.0$, $E = 9500$, $E_{05} = 6500$

Width	Depth
$L_b = 4300\,mm$	$L_d = 4300\,mm$
$C_{C_b} = \frac{4300}{76} = 56.58 > 50$, not ok	
Fails slenderness	

Check $(P_{rt})_b$

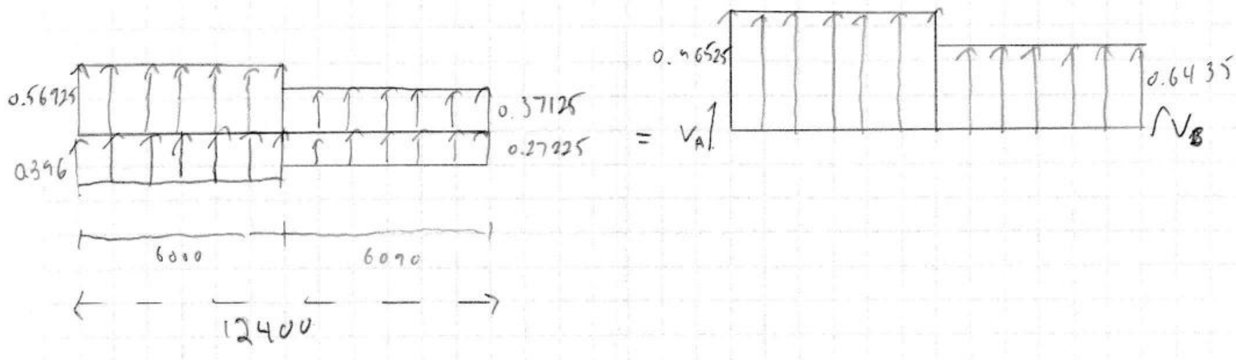
$C_{C,inf} = \frac{4300}{38} = 113.157 > 50$

does not govern

Diaphragm and Shearwall

The diaphragm and shearwall have been illustrated in the drawings shown before. See the calculations that went into these designs below:

To determine forces in shearwall, see south wall below w/ ULS loading



To find V_B

$$\Sigma M_A = 0, \quad 0.96525(6)\left(\tfrac{6}{2}\right)\left(\tfrac{5.24}{2}\right) + 0.6435(6.4)\left(\tfrac{5.24}{2}\right)\left(6 + \tfrac{6.4}{2}\right) - V_B(12.4)$$

$$V_B = 11.677 \times 1.4 = 16.347 \text{ kN}$$

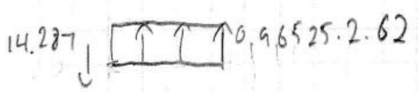
To find V_A

$$\Sigma F_N = 0, \quad 0.96525\left(\tfrac{5.24}{2}\right)(6) + 0.6435\left(\tfrac{5.24}{2}\right)(6.40) - 11.677$$

$$= 14.287 \times 1.4 = 20 \text{ kN (ULS)} \sim 20 \text{ kN}$$

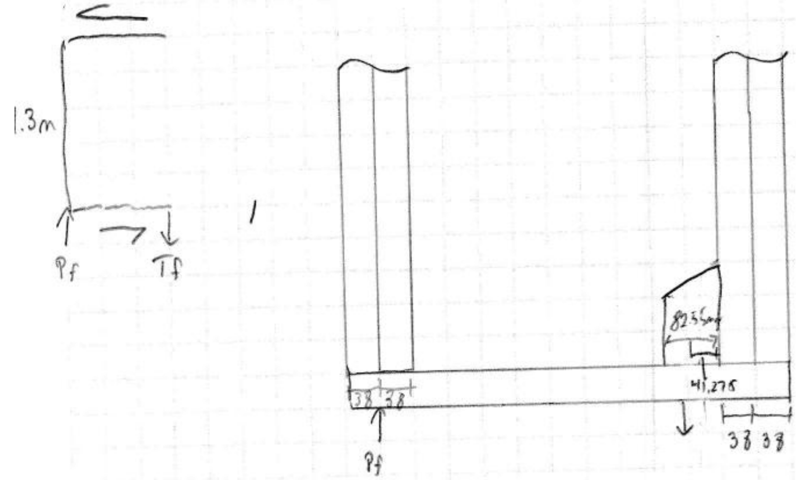
for SLS, $V_A = (20 \div 1.4) \times 0.75 = 10.72 \text{ kN}$

Point crossing axis



$$x = \frac{14.287}{0.96525 \cdot 2.62} = 5.65$$

To find shearwall chord forces



Lever arm = 3094 - 38 - (38×2 + 41.275)

= 2939 mm

$\therefore P_f = T_f = \dfrac{20 \cdot 4.3}{2.939} = 29.26$ kN = 6580 lbs

From WDM, $T_r = 1.15 \times 34.2 \cdot 2 = 78.6$ kN , 78.6 > 29.26, OK

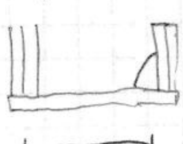
$P_r = \times 24.7 \times 2 = 49.4$ kN , 44.4 > 29.26, OK

^ didn't use 1.15 bc passed w/o it.

∴ Use double stud. However, holddowns for this Tf require triple studs.

To attempt to avoid this, forces are assumed distributed to other 2 segments along line ①.

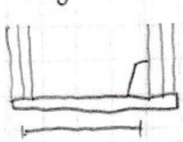
Segment 1



2938 mm

$T_f = \dfrac{V_1 \cdot 4.3}{2.938}$

Segment 2

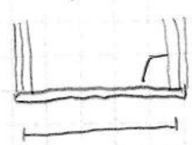


1360 - 38 - (38×2 + 41.275)

= 1205 mm

$T_f = \dfrac{V_2 \cdot 4.3}{1.205}$

Segment 3



1220 - 38×3 - 41.275

= 1065 mm

$T_f = \dfrac{V_3 \cdot 4.3}{1.065}$

From iterative distribution calculation (See Excel), Segment 1 takes 6.05 kN, 2 takes 2.48 kN, and Segment 3 takes 2.18 kN (SLS) ∴ Total SLS load = 10.71 kN

T_f (Segment 1) = $\frac{\frac{6.05}{10.71} \cdot 20 \cdot 4.3}{2.038} = 16.54$, T_f (Segment 2) = $\frac{\frac{2.48}{10.71} \cdot 20 \cdot 4.3}{1.205} = 16.53$, T_f (Segment 3) = $\frac{\frac{2.18}{10.71} \cdot 20 \cdot 4.3}{1.065} = 16.44$

∴ Worst case = 16.54 kN = 3720 lbs

As before, double stud works. HD5B holddowns can be assumed, which have 3785 lbs capacity for a member w/ 3" width (double stud) and SPF. Deflections at highest load (3785) are 3.96 mm, or 0.156 in.

Segment 1: $\frac{6.05}{10.71} \cdot 20 = 11.30$ kN. This is the shear anchor to concrete connection shear must resist.

$k_D = 1.15$
$k_{D_g} = 1$
$k_{sp} = 1$
$k_{sv} = 1$
$k_{st} = 1$
$k_T = 1$

Capacity of 3 lag screws in 1 row, spaced 152 mm apart w/ 1/2" m

$14.2 \cdot 1.15 = 16.33 > 11.3$, ∴ Use spacing

(WDM pg. 471)

↳ 127 mm apart could work in this case. However, larger spacing is more conservative, and will work in the case of more shear force present on opposite shearwall.

Forces (kN)	
ULS	20
SLS	10.71
Sheathing to framing connection	
KD	1.15
KSF	1
KT	1
nu*phi (WDM 12.5mm CSP, with 3.5" nails 4.11mmø)	0.665
Nu (nu*phi*KD)	0.76475
Spacing on edge (m)	0.15
Spacing intermediate support (m)	0.3
vd (kN/m)	5.09833333
ns	1
Jus	1
Js	1
Jhd	1
Jd	1.3
LD (m)	3.094
Vr (kN)	20.5065163
<u>Panel Buckling not evaluated (does not govern, as illustrated with diaphragm calculation)</u>	

Initial Guess - Shear Distributi Section 1		Section 2	Section 3	Notes
V (kN)	5.836666667	2.568888889	2.304444444	Initial assumption based on total force (10.71 kN)x[(Segment L)/(Total L)]
v (kN/m)	1.888888889	1.888888889	1.888888889	
Hs (m)	4.3	4.3	4.3	
EA	101080000	101080000	101080000	Uses E=9500 Mpa,
Ls (m)	3.09	1.36	1.22	
Bv	6900	6900	6900	See Table 9.1
en	0.047546085	0.047546085	0.047546085	Based on 3.5" nails with 4.11mm diameter
da anchor rod	0.988097113	0.434890639	0.390122485	Assumes 1/2" diameter
da holddown (assumes HD5B)	0.836048941	0.36796976	0.33009052	
Deflections (mm)				
Bending	0.320550757	0.728310176	0.811886754	
Shear	1.177133655	1.177133655	1.177133655	
Nail slip	0.51112041	0.51112041	0.51112041	
da	2.538455673	2.538455673	2.538455673	
Total Deflection	4.547260495	4.955019914	5.038596492	

Iterative Distribution - Displac Section 1		Section 2	Section 3	
V (kN)	6.05	2.48	2.18	Initial assumption based on total force (10.71 kN)x[(Segment L)/(Total L)]
v (kN/m)	1.957928803	1.823529412	1.786885246	
Hs	4.3	4.3	4.3	
EA	101080000	101080000	101080000	Uses E=9500 Mpa,
Ls	3.09	1.36	1.22	
Bv	6900	6900	6900	See Table 9.1
en	0.051085274	0.044312625	0.042549578	Based on 3.5" nails with 4.11mm diameter
da anchor rod	0.656944444	0.269292929	0.236717172	Assumes 1/2" diameter
da holddown (assumes HD5B)	0.953267681	0.390760967	0.343491495	
Deflections (mm)				
Bending	0.332267061	0.703109132	0.768043303	
Shear	1.220158529	1.136402387	1.113566168	
Nail slip	0.549166694	0.476360722	0.457407968	
da	2.240748266	2.086935113	2.04499776	
Total Deflection (mm)	4.34234055	4.402807353	4.384015198	

Diaphragm deflection is 1 mm. Therefore, total deflection is 4.4mm+1mm=5.4mm
Since H/500=5240/500=10.5mm, sufficient against deflections

1	Initial Guess - Shear Distributed Based on L/Total Length	Section 1	Section 2	Section 3
2	V (kN)	=10.71*(B6/SUM($B$6:$D$6))	=10.71*(C6/SUM($B$6:$D$6))	=10.71*(D6/SUM($B$6:$D$6))
3	v (kN/m)	=B2/B6	=C2/C6	=D2/D6
4	Hs (m)	4.3	4.3	4.3
5	EA	=9500*38*140*2	=9500*38*140*2	=9500*38*140*2
6	Ls (m)	3.09	1.36	1.22
7	Bv	6900	6900	6900
8	en	=(0.013*B3*150/(4.11^2))^2	=(0.013*C3*150/(4.11^2))^2	=(0.013*D3*150/(4.11^2))^2
9	da anchor rod	=B2*4.3/(200000*127)*1000^2	=C2*4.3/(200000*127)*1000^2	=D2*4.3/(200000*127)*1000^2
10	da holddown (assumes HD5B)	=3.6*B2/25.1325	=3.6*C2/25.1325	=3.6*D2/25.1325
11	*Deflections (mm)*			
12	Bending	=(2*B3*B4^3)/(3*B5*B6)*1000*1000	=(2*C3*C4^3)/(3*C5*C6)*1000*1000	=(2*D3*D4^3)/(3*D5*D6)*1000*1000
13	Shear	=B3*B4/B7*1000	=C3*C4/C7*1000	=D3*D4/D7*1000
14	Nail slip	=0.0025*B4*B8*1000	=0.0025*C4*C8*1000	=0.0025*D4*D8*1000
15	da	=(B4/B6)*SUM(B9:B10)	=(C4/C6)*SUM(C9:C10)	=(D4/D6)*SUM(D9:D10)
16	Total Deflection	=SUM(B12:B15)	=SUM(C12:C15)	=SUM(D12:D15)
17				
18				
19	Iterative Distribution - Displacement is the same	Section 1	Section 2	Section 3
20	V (kN)	6.05	2.48	=10.71-B20-C20
21	v (kN/m)	=B20/B24	=C20/C24	=D20/D24
22	Hs	4.3	4.3	4.3
23	EA	=9500*38*140*2	=9500*38*140*2	=9500*38*140*2
24	Ls	3.09	1.36	1.22
25	Bv	6900	6900	6900
26	en	=(0.013*B21*150/(4.11^2))^2	=(0.013*C21*150/(4.11^2))^2	=(0.013*D21*150/(4.11^2))^2
27	da anchor rod	=B20*4.3/(200000*198)*1000^2	=C20*4.3/(200000*198)*1000^2	=D20*4.3/(200000*198)*1000^2
28	da holddown (assumes HD5B)	=3.96*B20/25.1325	=3.96*C20/25.1325	=3.96*D20/25.1325
29	*Deflections (mm)*			
30	Bending	=(2*B21*B22^3)/(3*B23*B24)*1000*1000	=(2*C21*C22^3)/(3*C23*C24)*1000*	=(2*D21*D22^3)/(3*D23*D24)*1000*100
31	Shear	=B21*B22/B25*1000	=C21*C22/C25*1000	=D21*D22/D25*1000
32	Nail slip	=0.0025*B22*B26*1000	=0.0025*C22*C26*1000	=0.0025*D22*D26*1000
33	da	=(B22/B24)*SUM(B27:B28)	=(C22/C24)*SUM(C27:C28)	=(D22/D24)*SUM(D27:D28)
34	Total Deflection (mm)	=SUM(B30:B33)	=SUM(C30:C33)	=SUM(D30:D33)

Holdowns (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

Model No.	Material		Dimensions (in.)							Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load	Code Ref.
	Base (in.)	Body (ga.)	HB	SB	W	H	B	CL	SO	Anchor Dia. Bolt	Stud Bolts		DF/SP	SPF/HF		
HD3B	—	12	4¾	2½	2½	8⅝	2¼	1 5⁄16	⅜	⅝	(2) ⅝	1½ x 3½	1,895	1,610	0.156	
												2½ x 3½	2,525	2,145	0.169	
												3 x 3½	3,130	3,050	0.12	
												3½ x 3½	3,130	3,050	0.12	
HD5B	3⁄16	10	5¼	3	2½	9⅜	2½	1¼	2	⅝	(2) ¾	1½ x 3½	2,405	2,070	0.153	
												2½ x 3½	3,750	3,190	0.129	
												3 x 3½	4,505	3,785	0.156	
												3½ x 3½	4,935	4,195	0.15	
HD7B	3⁄16	10	5¼	3	2½	12⅜	2½	1¼	2	⅞	(3) ¾	3 x 3½	6,645	5,650	0.142	
												3½ x 3½	7,310	6,215	0.154	
												3½ x 4½	7,345	6,245	0.155	
HD9B	⅜	7	6⅛	3½	2⅞	14	2½	1¼	2⅜	⅞	(3) ⅞	3½ x 3½	7,740	6,580	0.159	
												3½ x 4½	9,920	8,430	0.178	IBC, FL, LA
												3½ x 5½	9,920	8,430	0.178	

Coded | Stays | Iterate

Shearwall & Unblocked Diaphragm

Factored	Pressure	KN/m	Kpa	Length	m
2.360358	1	0.972675	0.37125	L	12.4
	4	0.713295	0.27225	y	6
3.540537	1e	1.491435	0.56925	z	1
	4e	1.03752	0.396	h	2.62

	sum x	36.34951
Shear	R	20.00213
Vf	L	16.34738
		36.34951

Unblocked Diaphragm:
"Blocking all panel edges adds cost and labour to the construction process and can complicate venting of roof insulation"

FULL LENGTH

Diaphragm Shear

Vrs = Phi vd JD Js Jf Jud LD

	Phi	0.8	Nu	1.043932
	vd	6.959546	KD	1.15
	JD	1.3	nu	0.907767
	Js	1	edg. Space	0.15
table 11.1	Jf	0.89		
table 11.2	Jud	0.67		
Length oth	LD	13.601		

Assume:
15.5mm CSP Plywood
3.5" 4.11 dia. Nails
150mm around panel edges 300mm for intermediate supports

Nr'ns	0.881	pg377
nu	1.10125	

Vrs	58.7016 >	Vf	20.00213

Panel Buckling Check

Vrs = Phi vpb KD KS KT LD

	Eta	0.25556	KD	1.15
	Alpha	2.123647	LD	13.601
			t	15.5
			Phi	0.8
table 9.2	Ba,90	75000		
Based on p	Ba,0	59000		
	Bv	8500		
	a	2400		
	b	1200		
	Kpb	1.059657		
	vpb	49.29863		

b x a
Assume 1200x2400
i.e. 4x8'

Vrs	616.8698 >	Vf	20.00213

SOUTH SHEARWALL (SMALL)

Passes -> Other larger segments therefore must pass

Diaphragm Shear

Vrs = Phi vd JD Js Jf Jud LD

	Phi	0.8	Nu	1.043932
	vd	10.43932	KD	1.15
	JD	1.3	nu	0.907767
	Js	1	Int. Space	0.1
table 11.1	Jf	0.89		
table 11.2	Jud	0.67		
Length oth	LD	3.1		

Assume:
15.5mm CSP Plywood
3.5" 4.11 dia. Nails
150mm around panel edges 300mm for intermediate supports

Nr'ns	0.881	pg377
nu	1.10125	

Utilization :D

Vrs	20.06929 >	Vf	20.00213	0.996654

Panel Buckling Check

Vrs = Phi vpb KD KS KT LD

	Eta	0.25556	KD	1.15
	Alpha	2.123647	LD	3.1
			t	15.5
			Phi	0.8
table 9.2	Ba,90	75000		
Based on p	Ba,0	59000		
	Bv	8500		
	a	2400		
	b	1200		
	Kpb	1.059657		
	vpb	49.29863		

b x a
Assume 1200x2400
i.e. 4x8'

Vrs	140.5997 >	Vf	20.00213

Chord Forces

Tension check, nail @ splices

Assume:
Same size nails to cut down on errors on site
Clinched due to length

x	5.649462	Pf/Tf	4.216151
Moment	56.50064		
Length - 1/	13.401		
Tr	39.33 >	Tf, Pf	

2x6 Tr from design table x KD

Nailing for top chord splices:

Phi	0.8
nu	1.159945
KD	1.15
Nu	1.333937
JD	1.3
JB	1.6
JF	2.08
Nr	2.219671
nf	2

Diaphragm Deflection

	v	0.787841
	L	12.4
2x6	EA Chord	50540000
	LD	13.601
	Bv	8500
sheath-frai	en	0.003676
Sum	Delta C * x	16648.09
	Delta d	1.040968

SLS

Note: Find blocked deflection, for an approx. square building it should be similar

v value and spec'd moment chang right to left

Splice	1	2
Specified diaphragm moment	22.5680681	27.46935
Specified force	1.684058511	2.049798
Nails each side of splice	2	2
Force/nail	842.0292554	1024.899
Delta C	0.83985396	1.244261
x	2800	4800
Delta C * x	2351.591089	5972.455

Chord Leng 4.8

f1	25.72752	f2	20.1369	f3	22.1317	df	4.11
t1	15.5	t2	73.4	fy	594.5	L	3.5

a	1638.972		
b	6074.779		
c	3037.39		
d	907.7669		
e	2132.237		
f	1542.75		
g	1159.945		Diaphragm Nail
ANS	907.7669		

G	0.42
f1	25.72752
f2	20.1369
f3	22.1317
fy	594.5

f1	25.72752	f2	20.1369	f3	22.1317	df	4.11
t1	38	t2	38	fy	594.5	L	3.5

a	4018.124		
b	3144.981		
c	1572.491		
d	1383.597		
e	1383.597		
f	1432.621		
g	1159.945		Top Chord Nail
ANS	1159.945		